Geotechnical Engineering Report

48 Bed Community RTF
16015 NE 50th Avenue
Vancouver, Washington

Prepared for:
Washington State Department of Social and Health Services
PO Box 45848
Olympia, Washington 98504

June 16, 2021
PBS Project 73200.011

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1 INTRODUCTION

1.1 General

This report presents results of PBS Engineering and Environmental Inc. (PBS) geotechnical engineering services for the proposed residential treatment facility located at 16015 NE 50th Avenue in Vancouver, Washington (site). The general site location is shown on the Vicinity Map, Figure 1. The locations of PBS' explorations in relation to existing site features are shown on the Site Plan, Figure 2.

1.2 Purpose and Scope

The purpose of PBS' services was to develop geotechnical design and construction recommendations in support of the planned buildings on the western portions of the site. This was accomplished by performing the following scope of services.

1.2.1 Literature and Records Review

PBS reviewed various published geologic maps of the area for information regarding geologic conditions and hazards at or near the site. PBS also reviewed previously completed reports for the project vicinity.

1.2.2 Subsurface Explorations

The site was explored by excavating 17 test pits to depths of approximately 10 feet below the existing ground surface (bgs), advancing three cone penetrometer test (CPT) probes to depths of 50 to 56 feet bgs, and advancing three borings to depths of 31.5 feet bgs within the proposed development footprint and surrounding pavement areas. The test pits and soil borings were logged and representative soil samples collected by a member of the PBS geotechnical engineering staff. The approximate boring, test pit and CPT locations are shown on the Site Plan, Figure 2. The interpreted boring, test pit and CPT logs are presented as Figures A1 through A23 in Appendix A, Field Explorations. Shear wave velocities were collected at 2-meter intervals in CPT-1 and are presented on Figure A24 in Appendix A.

1.2.3 Field Infiltration Testing

Cased-hole, falling-head field infiltration testing was completed in test pit TP-5 within the proposed development at a depth of approximately 10 feet bgs, and in test pits TP-7 and TP-8 at a depth of approximately 5 feet bgs. Infiltration testing was not completed in the borings due to the presence of groundwater at the test elevation. Infiltration testing was monitored by PBS geotechnical engineering staff.

1.2.4 Soils Testing

Soil samples were returned to our laboratory and classified in general accordance with the Unified Soil Classification System (ASTM D2487) and/or the Visual-Manual Procedure (ASTM D2488). Laboratory tests included natural moisture contents and grain-size analyses. Laboratory test results are included in the exploration logs in Appendix A, Field Explorations; and in Appendix B, Laboratory Testing.

1.2.5 Geotechnical Engineering Analysis

Data collected during the subsurface exploration, literature research, and testing were used to develop site-specific geotechnical design parameters and construction recommendations.

1.2.6 Report Preparation

This Geotechnical Engineering Report summarizes the results of our explorations, testing, and analyses, including information relating to the following:

- Field exploration logs and site plan showing approximate exploration locations
- Laboratory test results

- Groundwater considerations
- Long-term infiltration rates for shallow and deep infiltration facilities
- Liquefaction potential
- Shallow foundation design recommendations:
    - Minimum embedment
    - Allowable bearing pressure
    - Estimated settlement (total and differential)
    - Sliding coefficient
    - Foundation drainage recommendations
- Earthwork and grading, cut, and fill recommendations:
    - Structural fill materials and preparation, and reuse of on-site soils
    - Wet weather considerations
    - Utility trench excavation and backfill requirements
    - Temporary and permanent slope inclinations
- Seismic design criteria in accordance with the 2018 International Building Code (IBC) with state of Washington amendments
- Slab and pavement subgrade preparation recommendations
- Recommended asphalt concrete (AC) and portland cement concrete (PCC) pavement sections for parking and associated drive lanes

1.3 Project Understanding

PBS understands the client plans to construct up to three 18,000-square-foot buildings, each with associated parking and landscape areas. The new buildings will be located on approximately the western third of the 20-acre site. The site is relatively flat and covered predominantly with grass and a few trees around the existing residence and barn, and in the northeastern portion of the site.

PBS understands that the purchase of this property is currently in negotiation and the locations of the proposed building and associated development plans should be considered very preliminary.

2 SITE CONDITIONS

2.1 Surface Description

The site is located in Vancouver, Washington, and occupies a rectangular parcel. The site is bordered to the west by NE 50th Avenue and Vancouver iTech Preparatory Public School, to the north and east by neighboring open parcels of land and residential properties, and to the south by NE 159th Street. The site is currently occupied by a residence and barn. The majority of the site is covered by grass, along with a few trees on the northeastern end of the parcel and the space surrounding the residence and barn.

Based on available LiDAR data provided by the Washington Department of Natural Resources (WADNR), the site is relatively flat, with ground surface elevations through the site ranging from about 170 to 180 feet above mean sea level (amsl) (WADNR, 2020). Outside of the site, the ground surface is relatively flat in all directions.

2.2 Geologic Setting

2.2.1 Regional Geology

The Portland Basin and Willamette Valley form a tectonic depression within the physiographic province of the Puget-Willamette Lowland that separates the Cascade Range from the Coast Range, and extends from the Puget Sound in Washington to Eugene, Oregon (Yeats et al, 1996). The Puget-Willamette Lowland is situated along the Cascadia Subduction Zone (CSZ) where oceanic rocks of the Juan de Fuca Plate are subducting beneath the North American Plate, resulting in deformation and uplift of the Coast Range and volcanism in the Cascade Range. Northwest-trending faults accommodating clockwise rotation of the North American Plate are found throughout the Puget-Willamette lowland (Brocher et al., 2017; USGS, 2020).

The greater Portland Basin is underlain by Columbia River Basalt Group (CRBG) flows consisting of numerous fine-grained volcanic eruptions between approximately 17 million years ago (Ma) and 6 Ma from fissures located in eastern Oregon, eastern Washington, and western Idaho (Beeson et al., 1991). These fissures released thousands of square kilometers, inundating areas east of the Cascade Range and entering western Oregon through a Miocene gap in the Cascade Range (present day Columbia Gorge) before reaching the ocean. Magmatic compositions of the CRBG allow the flows to be subdivided into distinct formations that can be further divided into members-based geochemical, paleomagnetic, and lithological properties.

Numerous northwest-trending faults govern the topography within the basin. Uplift and down dropping of crustal blocks have created topographic high points by offsetting regional scale flood basalts and down dropping basement rocks, creating infilled depressions and sediment basins, and have historically generated accommodation space for the accumulation of volcanic flows entering the basin and overlying fluvial deposits. Of these deposits, the Pliocene Troutdale Formation is the most widespread unit within the basin overlying CRBG volcanic flows. These friable to moderately strong conglomerates, with minor interbeds of sandstone and claystone, consist of well-rounded CRBG clasts and other exotic metamorphic and plutonic clasts. Younger quaternary deposits have accumulated above these conglomerates.

Cyclical Pleistocene cataclysmic floods deposited sediments and recarved the landscape within the Portland Basin more than 40 times over a 3,000-year timespan (Burns and Coe, 2012). As floodwaters entered the basin from the Columbia River Gorge, they slowed, depositing suspended sediments and bed loads. Topographic highpoints within the basin deflected floodwaters and generated areas that were scoured and eroded into older sediments and bedrock. These geomorphic features dominate the modern-day landscape and are indistinguishable within the Portland Basin LiDAR data (WADNR 2020; DOGAMI, 2020).

2.2.2 Local Geology

The site is underlain by Pleistocene age sand and silt of the cataclysmic flood deposits from repeated glacial-outburst floods (O'Connor et al., 2016). These facies consist of unconsolidated light brown to light gray silt, clay, and fine to medium sand, and were deposited as thick sheets over older sediments throughout the Portland-Vancouver basin. The sand is composed of quartz, feldspar, and conspicuous muscovite with the coarser sand beds containing abundant dark volcanic rock fragments, indicating Columbia River provenance.

2.3 Subsurface Conditions

The site was explored by advancing three soil borings, designated B-1 through B-3, to depths of 31.5 feet bgs, excavating 17 test pits, designated TP-1 through TP-17, to depths of approximately 10 feet bgs, and advancing three cone penetration test (CPT) probes, designated CPT-1 through CPT-3, to depths of up to 56 feet bgs. CPT-1 was terminated at 56 feet bgs due to refusal. The drilling was performed by Western States Soil Conservation, Inc., of Hubbard, Oregon, using a track mounted CME-850 drill rig and hollow stem auger and mud rotary drilling techniques. The test pit excavation was performed by Dan J. Fischer Excavation, Inc., of

Forest Grove, Oregon, using a Hitachi Zaxis 40U equipped with a 24-inch toothed bucket. The CPT probes were advanced using a 20-ton truck, mounted with a Vertek CPT 10 cm^2 electric seismic piezo cone, owned and operated by Geotechnical Explorations, Inc., of Keizer, Oregon.

PBS has summarized the subsurface units as follows:

TOPSOIL:	A 12- to 24-inch-thick layer of brown, low plasticity silt with sand and sandy silt to silty sand with organics and fine sand was encountered in all explorations.
SILTY SAND (SM)/POORLY GRADED SAND with SILT (SP-SM):	Brown, gray to dark gray, and tan sand with varying amounts of silt was encountered throughout the site beneath the topsoil to roughly 31.5 feet bgs. The silt had low plasticity with occasional iron staining, and the sand was fine-grained and micaceous. The interbedded silt and sand extends to about 56 feet bgs based on data collected from the CPTs. Soils become increasingly stiff at 56 feet, resulting in termination of CPT-1 due to practical refusal.

2.4 Groundwater

Static groundwater was observed at depths of 1 to 3 feet bgs while drilling the borings and seepage was observed from depths of 0.5 to 2 feet bgs during test pit excavation completed in February. CPT pore pressure dissipation testing indicated groundwater at depths of approximately 6 to 8 feet bgs. Groundwater depths measured in the piezometers are included in Table 1.

Table 1. Measured Groundwater Depths

Date	Measured Groundwater Depth (feet bgs)		
	B-1	B-2	B-3
2/20/2021	1.3	1.2	2.0
3/8/2021	1.7	1.8	2.6
3/29/2021	2.4	2.1	2.8
4/20/2021	4.2	4.0	4.6

Please note that groundwater levels can fluctuate during the year depending on climate, irrigation season, extended periods of precipitation, drought, and other factors.

2.5 Infiltration Testing

2.5.1 Field Testing

PBS completed cased-hole, falling-head infiltration testing in test pit TP-5 at a depth of approximately 10 feet bgs and in test pits TP-7 and TP-8 at a depth of approximately 5 feet bgs using cased-hole infiltration testing procedures. Shallow infiltration testing was completed near the locations of B-1, B-2, and B-3 at depths of approximately 15 inches, using open-hole test procedures. During testing, the 6-inch diameter casing or open-hole was filled with water to achieve a minimum 1-foot-high column of water. After a period of saturation, the height of the water column in the casing/hole was then measured initially and at regular, timed intervals. Results of our field infiltration testing are presented in Table 2.

Table 2. Infiltration Test Results

Test Location	Depth (feet bgs)	Field Measured Infiltration Rate (in/hr)	Design Infiltration Rate (in/hr)[a]	Soil Classification	USDA Hydrologic Soil Group
TP-5	10.0	6.2	1.55	Poorly Graded SAND with SILT (SP-SM)	A
TP-7	5.0	1.0	0.25	Silty SAND (SM)	C
TP-8	5.0	1.7	0.43	Silty SAND (SM)	B
B-1**	1.25	1.0	0.25	Sandy SILT (ML-SM)	C
B-2**	1.25	0.97	0.24	Sandy SILT (ML-SM)	C
B-3**	1.25	0.84	0.21	Sandy SILT (ML-SM)	C

[a] The recommended design infiltration rate is based on a correction factor of 4, which can also be reported as 0.25.
** Infiltration testing completed in shallow, hand-dug explorations near the indicated boring location.

The infiltration rates listed in Table 2 are not permeabilities/hydraulic conductivities, but field-measured rates, and do not include correction factors related to long-term infiltration rates. The design engineer should determine the appropriate correction factors to account for the planned level of pre-treatment, maintenance, vegetation, siltation, etc. Field-measured infiltration rates are typically reduced by a minimum factor of 2 to 4 for use in design.

Soil types can vary significantly over relatively short distances. The infiltration rate noted above is representative of one discrete location and depth. Installation of infiltration systems within the layer the field rate was measured is considered critical to proper performance of the systems.

2.5.2 USDA Hydrologic Soil Group Classification

The United States Department of Agriculture (USDA) categorizes soils in four hydrologic soil groups, A through D, and are designated mainly by particle size and hydraulic conductivity. Table 3 below shows general characteristics of each group as they are identified by the USDA.

Table 3. USDA Hydrologic Soil Group Parameters

Soil Properties	Hydrologic Soil Group			
	A	B	C	D
Saturated Hydraulic Conductivity (k) (inches/hour)	k>5.67	1.42<k<5.67	0.14<k<1.42	k<0.14

2.5.3 Laboratory Testing for Stormwater Treatment

The stormwater design will consider treatment from the existing site soils, which is a function of laboratory test results. PBS subcontracted laboratory testing for organic content, pH, and cation exchange capacity (CEC) of site soils in the vicinity of B-1, B-2, and B-3. Laboratory test results are summarized in Table 4.

Table 4. Laboratory Test Results For Stormwater Treatment

Boring	Organic Content (%)	pH	CEC (meq/100g)
B-1	1.3%	5.8	6.2
B-2	1.9%	5.8	5.4
B-3	3.6%	5.8	4.8

3 CONCLUSIONS AND RECOMMENDATIONS

3.1 Geotechnical Design Considerations

The project site is underlain by zones of medium dense, saturated, potentially liquefiable sand containing variable amounts of silt. Support on a shallow mat foundation is not feasible without some consideration of risk.

The following sections provide a more detailed discussion of our analysis and recommendations.

3.1.1 Liquefaction Potential

Liquefaction is defined as a decrease in the shear resistance of loose, saturated, cohesionless soil (e.g., sand) or low plasticity silt soils, due to the buildup of excess pore pressures generated during an earthquake. This results in a temporary transformation of the soil deposit into a viscous fluid. Liquefaction can result in ground settlement, foundation bearing capacity failure, and lateral spreading of ground.

Based on a review of the Liquefaction Susceptibility Map of Clark County, Washington (Palmer, 2004), the site is shown as having a very low liquefaction hazard. However, based the results of our analyses, 2 to 3 inches of total liquefaction settlement and 1 to 2 inches of differential liquefaction settlement could occur as the result of a code-based earthquake.

3.2 Code-Based Seismic Design Parameters

The current seismic design criteria for this project are based on the 2018 IBC. Due to the potential for liquefaction of site soils, the site should be considered Site Class F. However, in accordance with ASCE 7-16, for structures having a fundamental period of less than 0.5 second, a site-response analysis is not required to determine the spectral accelerations of liquefied soils and seismic design parameters can be determined using the pre-liquefaction site class, Site Class D. The seismic design criteria, in accordance with the 2018 IBC, are summarized in Table 5.

Table 5. 2018 IBC Seismic Design Parameters

Parameter	Short Period	1 Second
Maximum Credible Earthquake Spectral Acceleration	S_s = 0.79 g	S_1 = 0.37 g
Site Class	D*	
Site Coefficient	F_a = 1.18	F_v = 1.93 **
Adjusted Spectral Acceleration	S_{MS} = 0.94 g	S_{M1} = ***
Design Spectral Response Acceleration Parameters	S_{DS} = 0.63 g	S_{D1} = ***
MCE_G peak ground acceleration	PGA = 0.36 g	
Site amplification factor at PGA	F_{PGA} = 1.24	

Parameter	Short Period	1 Second
Site modified peak ground acceleration	PGA_M = 0.44 g	

g= Acceleration due to gravity

* Site Class D can be used if the fundamental period of the new structure is less than 0.5 second.

** This value of F_v shall only be used to calculate T_s

*** Site-specific site response analysis is not required for structures on Site Class D sites with S_1 greater than or equal to 0.2, provided the value of the seismic response coefficient C_s is determined by Eq. (12.8-2) for values of $T \leq 1.5T_s$ and taken as equal to 1.5 times the value computed in accordance with either Eq. (12.8-3) for $T_L \geq T > 1.5T_s$ or Eq. (12.8-4) for $T > T_L$.

3.3 Mat Foundation

The risk of surface manifestation of liquefaction is reduced by the presence of a non-liquefiable layer at the surface (i.e., “crust”). The crust could be composed of soils that are not susceptible to liquefaction or soils that are not saturated. The crust at this site consists only of unsaturated soils, corresponding to a crust thickness equal to the depth of groundwater, which ranges from 1 to 8 feet across the site. Using the estimated ground surface acceleration associated with a code-based earthquake, methods developed by Ishihara (1985), and the liquefiable layer thickness at the site, the crust would need to be on the order of 30 feet thick to limit the risk of surface manifestation of liquefaction at the site. Use of a mat foundation would help reduce the impacts of possible differential settlement resulting from liquefaction. The presence of the crust does not reduce the risk of liquefaction below that depth and 2 to 3 inches of liquefaction settlement would still occur at the site. Mat foundations should be designed to span a distance of at least 20 feet in the case of surface manifestation of liquefaction and loss of ground support. Specific recommendations for design and construction are included in the following sections.

3.3.1 Design Bearing Pressure

Mat foundations can be designed using a maximum allowable bearing pressure of 1,000 pounds per square foot (psf). The recommended allowable bearing pressure applies to the total of dead plus long-term live loads. Allowable bearing pressures may be increased by one-third for seismic and wind loads.

Foundations will settle in response to column and wall loads. Based on our evaluation of the subsurface conditions and our analysis, we estimate post-construction static settlement will be less than approximately 1 inch. Differential settlement will be on the order of one-half of the total settlement.

3.3.2 Allowable Bearing Capacity

PBS understands that the structural analyses will consider increased applied loads at the edge of a hypothetical sand boil over which the mat would be designed to free-span. For this relatively short-duration loading condition, considering a minimum depth of 2 feet and a bearing width of 4 feet, we recommend using an allowable bearing capacity of 5,000 psf. This includes a FS of 2.

3.3.3 Foundation Embedment Depth

PBS recommends that the perimeter of mat foundations be founded a minimum of 18 inches below the lowest adjacent grade. This can be accomplished with a thickened edge if the mat thickness is less than 18 inches.

3.3.4 Foundation Preparation

Excavations for foundations should be carefully prepared to a neat and undisturbed state. A representative from PBS should confirm suitable bearing conditions and evaluate all exposed footing subgrades. Observations should also confirm that loose or soft materials have been removed from new footing excavations and concrete slab-on-grade areas. Localized deepening of the excavations may be required to penetrate loose, wet, or deleterious materials. We suggest recompacting the exposed subgrade prior to forming and pouring concrete footings.

Satisfactory subgrade support for building mat foundations can be obtained from the on-site soil subgrade prepared in accordance with our recommendations presented in the Site Preparation, Wet/Freezing Weather and Wet Soil Conditions, and Imported Granular Materials sections of this report. A minimum 6-inch-thick layer of imported granular material should be placed and compacted over the prepared subgrade. Thicker aggregate sections may be necessary where undocumented fill is present, loose soils are present at subgrade elevation, and/or during wet conditions. Imported granular material should be composed of crushed rock or crushed gravel that is relatively well graded between coarse and fine, contains no deleterious materials, has a maximum particle size of 1½ inch, and has less than 5 percent by dry weight passing the US Standard No. 200 Sieve.

Mats supported on a subgrade and base course prepared in accordance with the preceding recommendations may be designed using a modulus of subgrade reaction (k) of 100 pounds per cubic inch (pci).

3.3.5 Lateral Resistance

Lateral loads can be resisted by passive earth pressure on the sides of the mat and by friction on the base of the mat. A passive earth pressure of 250 pounds per cubic foot (pcf) may be used for footings confined by native soils and new structural fills. The allowable passive pressure has been reduced by a factor of two to account for the large amount of deformation required to mobilize full passive resistance. Adjacent floor slabs, pavements, or the upper 12-inch depth of adjacent unpaved areas should not be considered when calculating passive resistance. For footings supported on native soils or new structural fills, use a coefficient of friction equal to 0.35 when calculating resistance to sliding. These values do not include a factor of safety (FS).

3.4 Ground Moisture

3.4.1 General

The perimeter ground surface and hard-scape should be sloped to drain away from all structures and away from adjacent slopes. Gutters should be tight-lined to a suitable discharge and maintained as free-flowing. All crawl spaces should be adequately ventilated and sloped to drain to a suitable, exterior discharge.

3.4.2 Building Pad Elevation/Fill

Due to the relatively shallow depth of groundwater at the site, we recommend that the finished floor elevation of new structures be at least 2 feet above the existing ground surface elevations. The bottom 12 inches of the fill should be clean, open-graded, 2- to 4-inch, angular drain rock, capped with 1½-inch-minus crushed rock.

3.4.3 Perimeter Footing Drains

Due to the relatively low permeability of site soils and the potential for perched groundwater at the site, we recommend perimeter foundation drains be installed around all proposed structures.

The foundation subdrainage system should include a minimum 4-inch diameter perforated pipe in a drain rock envelope. A non-woven geotextile filter fabric, such as Mirafi 140N or equivalent, should be used to completely wrap the drain rock envelope, separating it from the native soil and footing backfill materials. The invert of the perimeter drain lines should be placed approximately at the bottom of footing elevation. Also, the subdrainage system should be sealed at the ground surface. The perforated subdrainage pipe should be laid to drain by gravity into a non-perforated solid pipe and finally connected to the site drainage stem at a suitable location. Water from downspouts and surface water should be independently collected and routed to a storm sewer or other positive outlet. This water must not be allowed to enter the bearing soils.

3.4.4 Vapor Flow Retarder

A continuous, impervious barrier must be installed over the ground surface in the crawl space and under slabs of all structures. Barriers should be installed per the manufacturer's recommendations.

3.5 Recommended Pavement Sections

The provided pavement recommendations were developed based on our experience with similar projects and references the associated Washington Department of Transportation (WSDOT) specifications for construction.

3.5.1 Asphalt Concrete (AC)

The minimum recommended AC pavement section thicknesses are provided in Table 6. These pavement recommendations were developed considering a resilient modulus of 6,000 psi (CBR=4) and a 20-year design life.

Table 6. Minimum AC Pavement Sections

Traffic Loading	AC (inches)	Base Course (inches)	Subgrade
Pull-in Car Parking Only	3	9	Firm subgrade as verified by PBS personnel*
Drive Lanes and Access Roads	4	12	

* Subgrade must pass proofroll

The asphalt cement binder should be selected following WSDOT SS 9-02.1(4) – Performance Graded Asphalt Binder. The AC should consist of ½-inch hot mix asphalt (HMA) with a maximum lift thickness of 3 inches. The AC should conform to WSDOT SS 5-04.3(7)A – Mix Design, WSDOT SS 9-03.8(2) – HMA Test Requirements, and WSDOT SS 9-03.8(6) – HMA Proportions of Materials. The AC should be compacted to 91 percent of the maximum theoretical density (Rice value) of the mix, as determined in accordance with ASTM D2041, following the guidelines set in WSDOT SS 5-04.3(10) – Compaction.

3.5.2 Portland Cement Concrete (PCC)

We understand that PCC pavements will be used for ADA parking stalls, sidewalks, and crosswalks at the site. Based on subsurface conditions encountered at the site and our experience with similar developments, we recommend that the pavement section consist of 6 inches of PCC over 6 inches of aggregate base course. Longitudinal and transverse joint spacing should not exceed 12 feet and 15 feet, respectively.

The recommended PCC pavement section is contingent on the following recommendations being implemented during construction.

- Adequate drainage should be provided at the surface such that the subgrade soils are not allowed to become saturated by infiltration of surface runoff.
- Concrete slumps should be between 3 and 4 inches. The concrete should be properly cured in accordance with Portland Cement Association (PCA) recommended procedures and vehicular traffic should not be allowed for 3 days (automobile traffic) or 7 days (truck traffic).
- Construction joint spacing should not exceed 12 feet.
- Over-finishing of concrete pavements should be avoided. Typically, a broom or burlap drag finish should be used.

3.5.3 Construction Considerations

Due to the presence of soft/wet surficial soils at the site, depending on final grades, subgrade stabilization using cement amendment may be an efficient way to improve the subgrade stiffness and reduce disturbance during construction, particularly during wet conditions. Cement amendment of pavement area subgrades would allow for a possible reduction of the pavement base course section.

Heavy construction traffic on new pavements or partial pavement sections (such as base course over the prepared subgrade) will likely exceed the design loads and could potentially damage or shorten the pavement life; therefore, we recommend construction traffic not be allowed on new pavements, or that the contractor take appropriate precautions to protect the subgrade and pavement during construction.

If construction traffic is to be allowed on newly constructed road sections, an allowance for this additional traffic will need to be made in the design pavement section. Depending on weather conditions at the time of construction, a thicker aggregate base course section could be required to support construction traffic during preparation and placement of the pavement section.

4 CONSTRUCTION RECOMMENDATIONS

4.1 Site Preparation

Construction of the proposed structures will involve clearing and grubbing of the existing vegetation or demolition of possible existing structures. Demolition should include removal of existing pavement, utilities, etc., throughout the proposed new development. Underground utility lines or other abandoned structural elements should also be removed. The voids resulting from removal of foundations or loose soil in utility lines should be backfilled with compacted structural fill. The base of these excavations should be excavated to firm native subgrade before filling, with sides sloped at a minimum of 1H:1V to allow for uniform compaction. Materials generated during demolition should be transported off site or stockpiled in areas designated by the owner's representative.

4.1.1 Proofrolling/Subgrade Verification

Following site preparation and prior to placing aggregate base over shallow foundation, floor slab, and pavement subgrades, the exposed subgrade should be evaluated either by proofrolling or another method of subgrade verification. The subgrade should be proofrolled with a fully loaded dump truck or similar heavy, rubber-tire construction equipment to identify unsuitable areas. If evaluation of the subgrades occurs during wet conditions, or if proofrolling the subgrades will result in disturbance, they should be evaluated by PBS using a steel foundation probe. We recommend that PBS be retained to observe the proofrolling and perform the subgrade verifications. Unsuitable areas identified during the field evaluation should be compacted to a firm condition or be excavated and replaced with structural fill.

4.1.2 Wet/Freezing Weather and Wet Soil Conditions

Due to the presence of fine-grained silt and sands in the near-surface materials at the site, construction equipment may have difficulty operating on the near-surface soils when the moisture content of the surface soil is more than a few percentage points above the optimum moisture required for compaction. Soils disturbed during site preparation activities, or unsuitable areas identified during proofrolling or probing, should be removed and replaced with compacted structural fill.

Site earthwork and subgrade preparation should not be completed during freezing conditions, except for mass excavation to the subgrade design elevations. We recommend the earthwork construction at the site be performed during the dry season.

Protection of the subgrade is the responsibility of the contractor. Construction of granular haul roads to the project site entrance may help reduce further damage to the pavement and disturbance of site soils. The actual thickness of haul roads and staging areas should be based on the contractors' approach to site development, and the amount and type of construction traffic. The imported granular material should be placed in one lift over the prepared undisturbed subgrade and compacted using a smooth-drum, non-vibratory roller. A geotextile fabric should be used to separate the subgrade from the imported granular material in areas of repeated construction traffic. Depending on site conditions, the geotextile should meet Washington State Department of Transportation (WSDOT) SS 9-33.2 – Geosynthetic Properties for soil separation or stabilization. The geotextile should be installed in conformance with WSDOT SS 2-12.3 – Construction Geosynthetic (Construction Requirements) and, as applicable, WSDOT SS 2-12.3(2) – Separation or WSDOT SS 2-12.3(3) – Stabilization.

4.1.3 Compacting Test Pit Locations

The test pit excavations were backfilled using the excavator bucket and relatively minimal compactive effort; therefore, soft spots can be expected at these locations. We recommend that the relatively uncompacted soil be removed from the test pits to a depth of at least 3 feet below finished subgrade elevation in pavement areas and to full depth in building areas. The resulting excavation should be backfilled with structural fill.

4.1.4 Till Zone

An approximately 12- to 24-inch-deep agricultural till zone was observed in the explorations. The strength of till zone soils is difficult to predict due to its disturbed structure from regular cultivation over the years. This material is generally soft to medium stiff.

We recommend the upper 12 inches of the stripped subgrade be removed and replaced with structural fill or scarified and recompacted to a depth of 12 inches, as recommended for structural fill, within all proposed structural fill, pavement, and improvement areas and for a 5-foot margin beyond these areas, as well as in areas where less than 1 foot of cut is required. The native silt can be sensitive to small changes in moisture content and will be difficult, if not impossible, to compact adequately during wet weather. While scarification and compaction of the subgrade is the best option for subgrade improvement, this is likely only possible during extended dry periods and following moisture conditioning of the soil.

4.2 Excavation

The near-surface soils at the site can be excavated with conventional earthwork equipment. Sloughing and caving should be anticipated. All excavations should be made in accordance with applicable Occupational Safety and Health Administration (OSHA) and state regulations. The contractor is solely responsible for adherence to the OSHA requirements. Trench cuts should stand relatively vertical to a depth of approximately 4 feet bgs, provided no groundwater seepage is present in the trench walls. Open excavation techniques may be used provided the excavation is configured in accordance with the OSHA requirements, groundwater seepage is not present, and with the understanding that some sloughing may occur. Trenches/excavations should be flattened if sloughing occurs or seepage is present. Use of a trench shield or other approved temporary shoring is recommended if vertical walls are desired for cuts deeper than 4 feet bgs. If dewatering is used, we recommend that the type and design of the dewatering system be the responsibility of the contractor, who is in the best position to choose systems that fit the overall plan of operation.

4.3 Structural Fill

The extent of site grading is currently unknown; however, PBS estimates that cuts and fills will be on the order of 2 feet. Structural fill should be placed over subgrade that has been prepared in conformance with the Site Preparation and Wet/Freezing Weather and Wet Soil Conditions sections of this report. Structural fill material should consist of relatively well-graded soil, or an approved rock product that is free of organic material and debris, and contains particles not greater than 4 inches nominal dimension.

The suitability of soil for use as compacted structural fill will depend on the gradation and moisture content of the soil when it is placed. As the amount of fines (material finer than the US Standard No. 200 Sieve) increases, soil becomes increasingly sensitive to small changes in moisture content and compaction becomes more difficult to achieve. Soils containing more than about 5 percent fines cannot consistently be compacted to a dense, non-yielding condition when the water content is significantly greater (or significantly less) than optimum.

If fill and excavated material will be placed on slopes steeper than 5H:1V, these must be keyed/benched into the existing slopes and installed in horizontal lifts. Vertical steps between benches should be approximately 2 feet.

4.3.1 On-Site Soil

On-site soils encountered in our explorations are generally suitable for placement as structural fill during dry weather when moisture content can be maintained by air drying and/or addition of water. The fine-grained fraction of the site soils are moisture sensitive, and during wet weather, may become unworkable because of excess moisture content. In order to reduce moisture content, some aerating and drying of fine-grained soils may be required. The material should be placed in lifts with a maximum uncompacted thickness of approximately 8 inches and compacted to at least 92 percent of the maximum dry density, as determined by ASTM D1557 (modified proctor).

4.3.2 Imported Granular Materials

Imported granular material used during periods of wet weather or for haul roads, building pad subgrades, staging areas, etc., should be pit or quarry run rock, crushed rock, or crushed gravel and sand, and should meet the specifications provided in WSDOT SS 9-03.14(2) – Select Borrow. In addition, the imported granular material should be fairly well graded between coarse and fine, and of the fraction passing the US Standard No. 4 Sieve, less than 5 percent by dry weight should pass the US Standard No. 200 Sieve.

Imported granular material should be placed in lifts with a maximum uncompacted thickness of 9 inches and be compacted to not less than 95 percent of the maximum dry density, as determined by ASTM D1557.

4.3.3 Base Aggregate

Base aggregate for floor slabs and beneath pavements should be clean crushed rock or crushed gravel. The base aggregate should contain no deleterious materials, meet specifications provided in WSDOT SS 9-03.9(3) – Crushed Surfacing Base Course, and have less than 5 percent (by dry weight) passing the US Standard No. 200 Sieve. The imported granular material should be placed in one lift and compacted to at least 95 percent of the maximum dry density, as determined by ASTM D1557.

4.3.4 Foundation Base Aggregate

Imported granular material placed at the base of excavations for spread footings, slabs-on-grade, and other below-grade structures should be clean, crushed rock or crushed gravel, and sand that is fairly well graded between coarse and fine. The granular materials should contain no deleterious materials, have a maximum particle size of 1½ inch, and meet WSDOT SS 9-03.12(1)A – Gravel Backfill for Foundations (Class A). The imported granular material should be placed in one lift and compacted to not less than 95 percent of the maximum dry density, as determined by ASTM D1557.

4.3.5 Trench Backfill

Trench backfill placed beneath, adjacent to, and for at least 2 feet above utility lines (i.e., the pipe zone) should consist of well-graded granular material with a maximum particle size of 1 inch and less than 10 percent by dry weight passing the US Standard No. 200 Sieve, and should meet the standards prescribed by WSDOT SS 9-03.12(3) – Gravel Backfill for Pipe Zone Bedding. The pipe zone backfill should be compacted to at least 90 percent of the maximum dry density as determined by ASTM D1557, or as required by the pipe manufacturer or local building department.

Within pavement areas or beneath building pads, the remainder of the trench backfill should consist of well-graded granular material with a maximum particle size of 1½ inches, less than 10 percent by dry weight passing the US Standard No. 200 Sieve, and should meet standards prescribed by WSDOT SS 9-03.19 – Bank Run Gravel for Trench Backfill. This material should be compacted to at least 92 percent of the maximum dry density, as determined by ASTM D1557, or as required by the pipe manufacturer or local building department. The upper 2 feet of the trench backfill should be compacted to at least 95 percent of the maximum dry density, as determined by ASTM D1557.

Outside of structural improvement areas (e.g., roadway alignments or building pads), trench backfill placed above the pipe zone should consist of excavated material free of wood waste, debris, clods, or rocks greater than 6 inches in diameter and meet WSDOT SS 9-03.14 – Borrow and WSDOT SS 9-03.15 – Native Material for Trench Backfill. This general trench backfill should be compacted to at least 90 percent of the maximum dry density, as determined by ASTM D1557, or as required by the pipe manufacturer or local building department.

4.3.6 Stabilization Material

Stabilization rock should consist of pit or quarry run rock that is well-graded, angular, crushed rock consisting of 4- or 6-inch-minus material with less than 5 percent passing the US Standard No. 4 Sieve. The material should be free of organic matter and other deleterious material. WSDOT SS 9-13.1(5) – Quarry Spalls can be used as a general specification for this material with the stipulation of limiting the maximum size to 6 inches.

5 ADDITIONAL SERVICES AND CONSTRUCTION OBSERVATIONS

In most cases, other services beyond completion of a final geotechnical engineering report are necessary or desirable to complete the project. Occasionally, conditions or circumstances arise that require additional work that was not anticipated when the geotechnical report was written. PBS offers a range of environmental, geological, geotechnical, and construction services to suit the varying needs of our clients.

PBS should be retained to review the plans and specifications for this project before they are finalized. Such a review allows us to verify that our recommendations and concerns have been adequately addressed in the design.

Satisfactory earthwork performance depends on the quality of construction. Sufficient observation of the contractor's activities is a key part of determining that the work is completed in accordance with the construction drawings and specifications. We recommend that PBS be retained to observe general excavation, stripping, fill placement, footing subgrades, and/or pile installation. Subsurface conditions observed during construction should be compared with those encountered during the subsurface explorations. Recognition of changed conditions requires experience; therefore, qualified personnel should visit the site with sufficient frequency to detect whether subsurface conditions change significantly from those anticipated.

6 LIMITATIONS

This report has been prepared for the exclusive use of the addressee, and their architects and engineers, for aiding in the design and construction of the proposed development and is not to be relied upon by other parties. It is not to be photographed, photocopied, or similarly reproduced, in total or in part, without express written consent of the client and PBS. It is the addressee's responsibility to provide this report to the appropriate design professionals, building officials, and contractors to ensure correct implementation of the recommendations.

The opinions, comments, and conclusions presented in this report are based upon information derived from our literature review, field explorations, laboratory testing, and engineering analyses. It is possible that soil, rock, or groundwater conditions could vary between or beyond the points explored. If soil, rock, or groundwater conditions are encountered during construction that differ from those described herein, the client is responsible for ensuring that PBS is notified immediately so that we may reevaluate the recommendations of this report.

Unanticipated fill, soil and rock conditions, and seasonal soil moisture and groundwater variations are commonly encountered and cannot be fully determined by merely taking soil samples or completing explorations such as soil borings or test pits. Such variations may result in changes to our recommendations and may require additional funds for expenses to attain a properly constructed project; therefore, we recommend a contingency fund to accommodate such potential extra costs.

The scope of work for this subsurface exploration and geotechnical report did not include environmental assessments or evaluations regarding the presence or absence of wetlands or hazardous substances in the soil, surface water, or groundwater at this site.

If there is a substantial lapse of time between the submission of this report and the start of work at the site, if conditions have changed due to natural causes or construction operations at or adjacent to the site, or if the basic project scheme is significantly modified from that assumed, this report should be reviewed to determine the applicability of the conclusions and recommendations presented herein. Land use, site conditions (both on and off site), or other factors may change over time and could materially affect our findings; therefore, this report should not be relied upon after three years from its issue, or in the event that the site conditions change.

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Important Information about This

Geotechnical-Engineering Report

Subsurface problems are a principal cause of construction delays, cost overruns, claims, and disputes.

While you cannot eliminate all such risks, you can manage them. The following information is provided to help.

The Geoprofessional Business Association (GBA) has prepared this advisory to help you – assumedly a client representative – interpret and apply this geotechnical-engineering report as effectively as possible. In that way, you can benefit from a lowered exposure to problems associated with subsurface conditions at project sites and development of them that, for decades, have been a principal cause of construction delays, cost overruns, claims, and disputes. If you have questions or want more information about any of the issues discussed herein, contact your GBA-member geotechnical engineer. Active engagement in GBA exposes geotechnical engineers to a wide array of risk-confrontation techniques that can be of genuine benefit for everyone involved with a construction project.

Understand the Geotechnical-Engineering Services Provided for this Report

Geotechnical-engineering services typically include the planning, collection, interpretation, and analysis of exploratory data from widely spaced borings and/or test pits. Field data are combined with results from laboratory tests of soil and rock samples obtained from field exploration (if applicable), observations made during site reconnaissance, and historical information to form one or more models of the expected subsurface conditions beneath the site. Local geology and alterations of the site surface and subsurface by previous and proposed construction are also important considerations. Geotechnical engineers apply their engineering training, experience, and judgment to adapt the requirements of the prospective project to the subsurface model(s). Estimates are made of the subsurface conditions that will likely be exposed during construction as well as the expected performance of foundations and other structures being planned and/or affected by construction activities.

The culmination of these geotechnical-engineering services is typically a geotechnical-engineering report providing the data obtained, a discussion of the subsurface model(s), the engineering and geologic engineering assessments and analyses made, and the recommendations developed to satisfy the given requirements of the project. These reports may be titled investigations, explorations, studies, assessments, or evaluations. Regardless of the title used, the geotechnical-engineering report is an engineering interpretation of the subsurface conditions within the context of the project and does not represent a close examination, systematic inquiry, or thorough investigation of all site and subsurface conditions.

Geotechnical-Engineering Services are Performed for Specific Purposes, Persons, and Projects, and At Specific Times

Geotechnical engineers structure their services to meet the specific needs, goals, and risk management preferences of their clients. A geotechnical-engineering study conducted for a given civil engineer will not likely meet the needs of a civil-works constructor or even a different civil engineer. Because each geotechnical-engineering study is unique, each geotechnical-engineering report is unique, prepared *solely* for the client.

Likewise, geotechnical-engineering services are performed for a specific project and purpose. For example, it is unlikely that a geotechnical-engineering study for a refrigerated warehouse will be the same as one prepared for a parking garage; and a few borings drilled during a preliminary study to evaluate site feasibility will not be adequate to develop geotechnical design recommendations for the project.

Do not rely on this report if your geotechnical engineer prepared it:

- for a different client;
- for a different project or purpose;
- for a different site (that may or may not include all or a portion of the original site); or
- before important events occurred at the site or adjacent to it; e.g., man-made events like construction or environmental remediation, or natural events like floods, droughts, earthquakes, or groundwater fluctuations.

Note, too, the reliability of a geotechnical-engineering report can be affected by the passage of time, because of factors like changed subsurface conditions; new or modified codes, standards, or regulations; or new techniques or tools. *If you are the least bit uncertain* about the continued reliability of this report, contact your geotechnical engineer before applying the recommendations in it. A minor amount of additional testing or analysis after the passage of time – if any is required at all – could prevent major problems.

Read this Report in Full

Costly problems have occurred because those relying on a geotechnical-engineering report did not read the report in its entirety. Do not rely on an executive summary. Do not read selective elements only. *Read and refer to the report in full.*

You Need to Inform Your Geotechnical Engineer About Change

Your geotechnical engineer considered unique, project-specific factors when developing the scope of study behind this report and developing the confirmation-dependent recommendations the report conveys. Typical changes that could erode the reliability of this report include those that affect:

- the site's size or shape;
- the elevation, configuration, location, orientation, function or weight of the proposed structure and the desired performance criteria;
- the composition of the design team; or
- project ownership.

As a general rule, *always* inform your geotechnical engineer of project or site changes – even minor ones – and request an assessment of their impact. *The geotechnical engineer who prepared this report cannot accept* *responsibility or liability for problems that arise because the geotechnical engineer was not informed about developments the engineer otherwise would have considered.*

Most of the "Findings" Related in This Report Are Professional Opinions

Before construction begins, geotechnical engineers explore a site's subsurface using various sampling and testing procedures. *Geotechnical engineers can observe actual subsurface conditions only at those specific locations where sampling and testing is performed.* The data derived from that sampling and testing were reviewed by your geotechnical engineer, who then applied professional judgement to form opinions about subsurface conditions throughout the site. Actual sitewide-subsurface conditions may differ – maybe significantly – from those indicated in this report. Confront that risk by retaining your geotechnical engineer to serve on the design team through project completion to obtain informed guidance quickly, whenever needed.

This Report's Recommendations Are Confirmation-Dependent

The recommendations included in this report – including any options or alternatives – are confirmation-dependent. In other words, they are not final, because the geotechnical engineer who developed them relied heavily on judgement and opinion to do so. Your geotechnical engineer can finalize the recommendations *only after observing actual subsurface conditions* exposed during construction. If through observation your geotechnical engineer confirms that the conditions assumed to exist actually do exist, the recommendations can be relied upon, assuming no other changes have occurred. *The geotechnical engineer who prepared this report cannot assume responsibility or liability for confirmation-dependent recommendations if you fail to retain that engineer to perform construction observation.*

This Report Could Be Misinterpreted

Other design professionals' misinterpretation of geotechnical-engineering reports has resulted in costly problems. Confront that risk by having your geotechnical engineer serve as a continuing member of the design team, to:

- confer with other design-team members;
- help develop specifications;
- review pertinent elements of other design professionals' plans and specifications; and
- be available whenever geotechnical-engineering guidance is needed.

You should also confront the risk of constructors misinterpreting this report. Do so by retaining your geotechnical engineer to participate in prebid and preconstruction conferences and to perform construction-phase observations.

Give Constructors a Complete Report and Guidance

Some owners and design professionals mistakenly believe they can shift unanticipated-subsurface-conditions liability to constructors by limiting the information they provide for bid preparation. To help prevent the costly, contentious problems this practice has caused, include the complete geotechnical-engineering report, along with any attachments or appendices, with your contract documents, *but be certain to note conspicuously that you've included the material for information purposes only.* To avoid misunderstanding, you may also want to note that "informational purposes" means constructors have no right to rely on the interpretations, opinions, conclusions, or recommendations in the report. Be certain that constructors know they may learn about specific project requirements, including options selected from the report, *only* from the design drawings and specifications. Remind constructors that they may perform their own studies if they want to, and *be sure to allow enough time* to permit them to do so. Only then might you be in a position to give constructors the information available to you, while requiring them to at least share some of the financial responsibilities stemming from unanticipated conditions. Conducting prebid and preconstruction conferences can also be valuable in this respect.

Read Responsibility Provisions Closely

Some client representatives, design professionals, and constructors do not realize that geotechnical engineering is far less exact than other engineering disciplines. This happens in part because soil and rock on project sites are typically heterogeneous and not manufactured materials with well-defined engineering properties like steel and concrete. That lack of understanding has nurtured unrealistic expectations that have resulted in disappointments, delays, cost overruns, claims, and disputes. To confront that risk, geotechnical engineers commonly include explanatory provisions in their reports. Sometimes labeled "limitations," many of these provisions indicate where geotechnical engineers' responsibilities begin and end, to help others recognize their own responsibilities and risks. *Read these provisions closely.* Ask questions. Your geotechnical engineer should respond fully and frankly.

Geoenvironmental Concerns Are Not Covered

The personnel, equipment, and techniques used to perform an environmental study – e.g., a "phase-one" or "phase-two" environmental site assessment – differ significantly from those used to perform a geotechnical-engineering study. For that reason, a geotechnical-engineering report does not usually provide environmental findings, conclusions, or recommendations; e.g., about the likelihood of encountering underground storage tanks or regulated contaminants. *Unanticipated subsurface environmental problems have led to project failures.* If you have not obtained your own environmental information about the project site, ask your geotechnical consultant for a recommendation on how to find environmental risk-management guidance.

Obtain Professional Assistance to Deal with Moisture Infiltration and Mold

While your geotechnical engineer may have addressed groundwater, water infiltration, or similar issues in this report, the engineer's services were not designed, conducted, or intended to prevent migration of moisture – including water vapor – from the soil through building slabs and walls and into the building interior, where it can cause mold growth and material-performance deficiencies. Accordingly, ***proper implementation of the geotechnical engineer's recommendations will not of itself be sufficient to prevent moisture infiltration. Confront the risk of moisture infiltration*** by including building-envelope or mold specialists on the design team. ***Geotechnical engineers are not building-envelope or mold specialists.***

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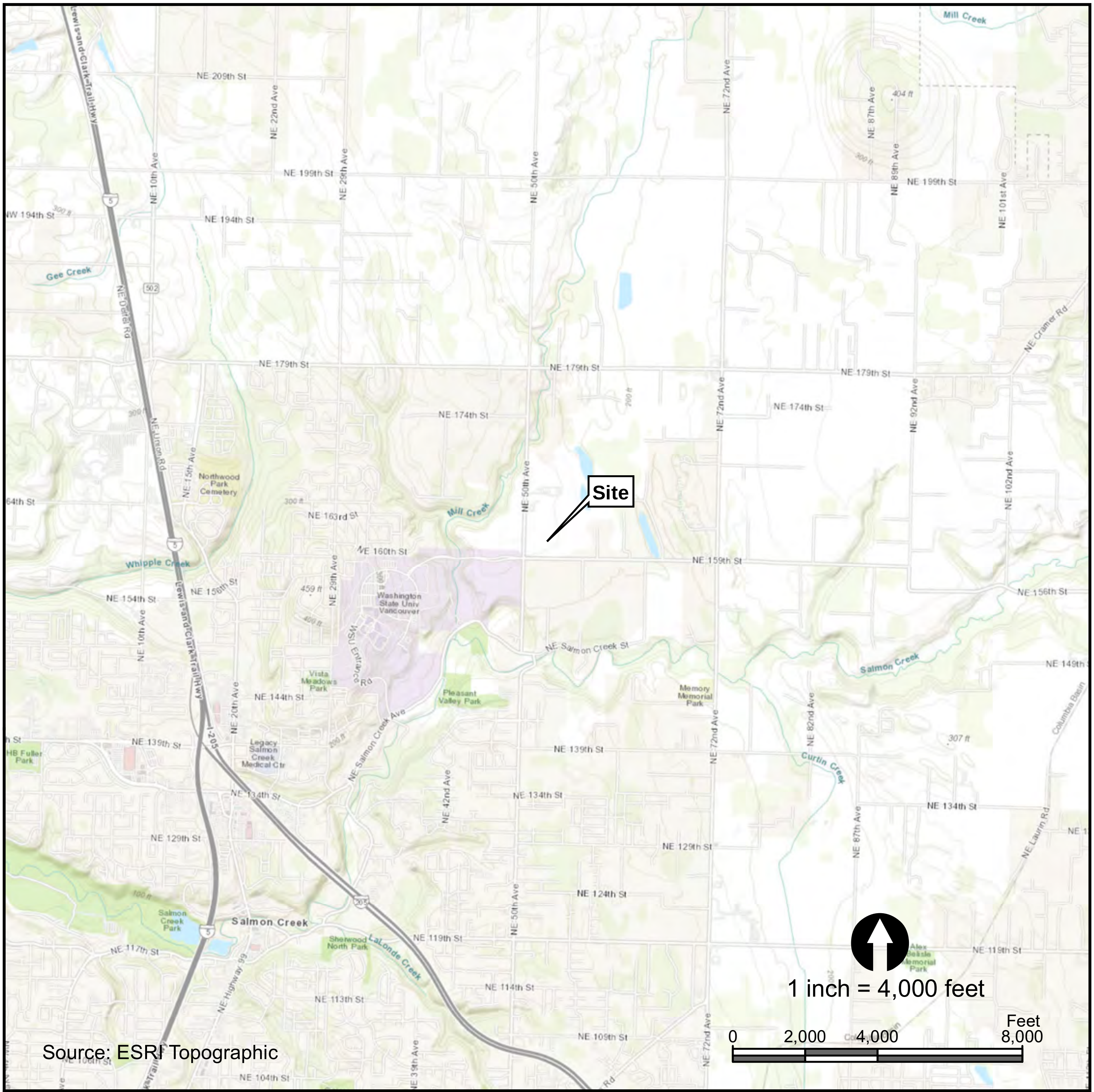
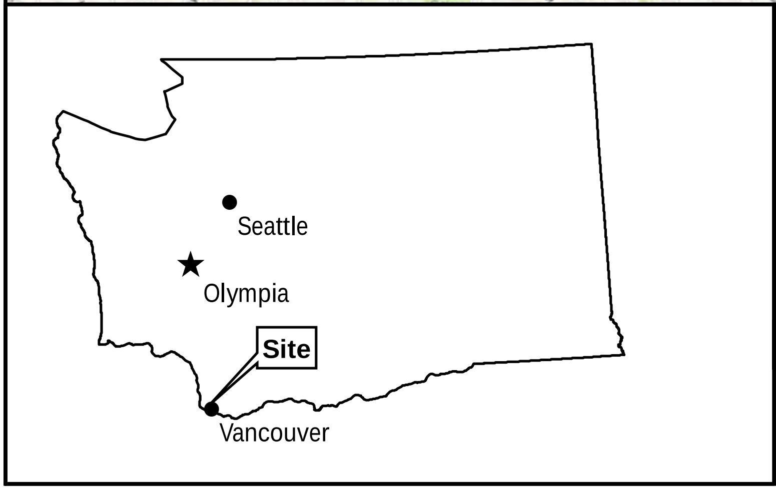
Figures





VICINITY MAP

48 BED COMMUNITY RTF
VANCOUVER, WASHINGTON

DATE: JUNE 2021 · PROJECT: 73200.011

PBS

FIGURE
1

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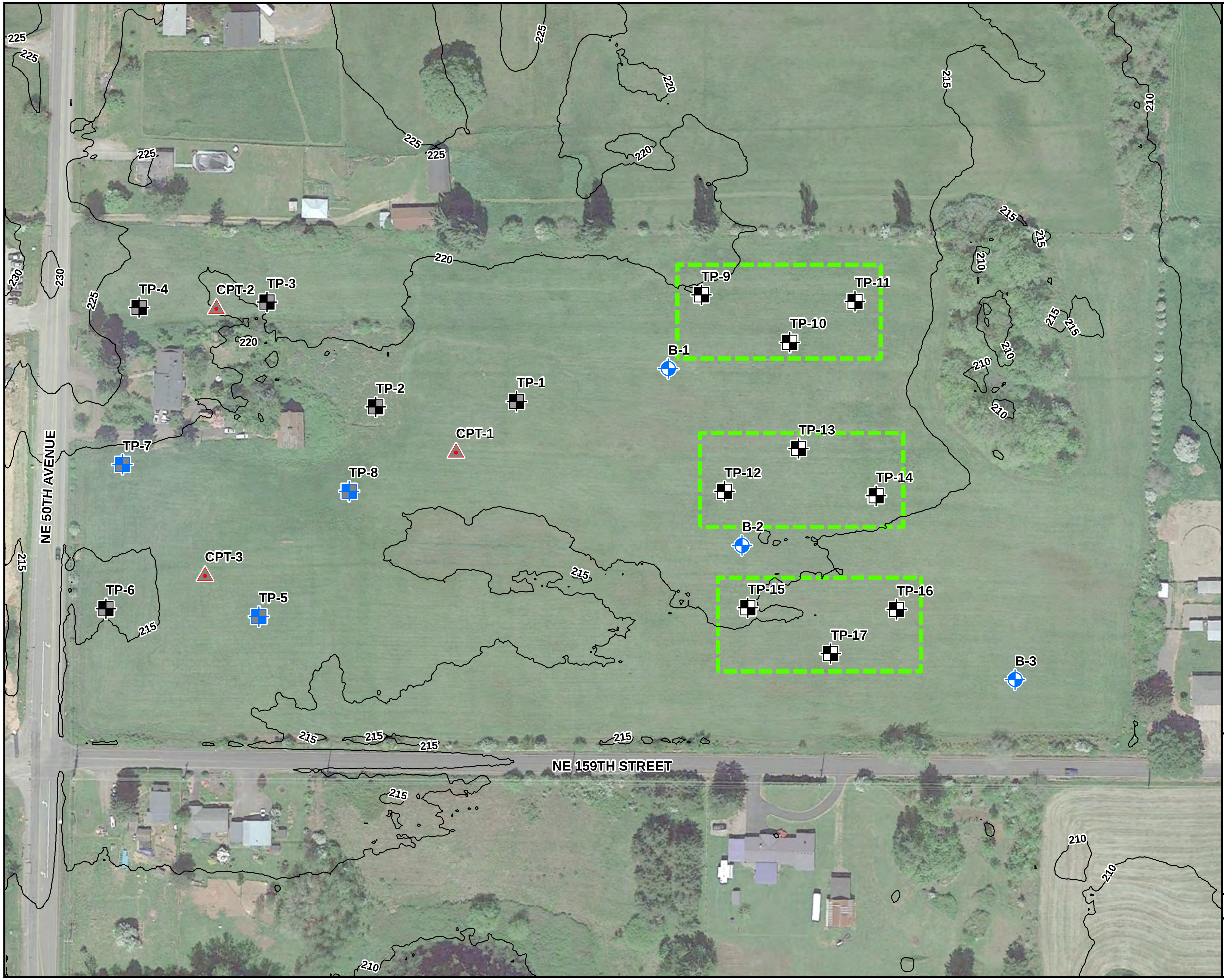

EXPLANATION

Additional Explorations

⊕ B-1 - Boring name and approximate location with infiltration test and piezometers installed upon completion

▚ TP-9 - Test pit name and approximate location

Previous Explorations

▚ TP-1 - Test pit name and approximate location

▚ TP-5 - Test pit name and approximate location with infiltration test

▲ CPT-1 - Cone penetration test name and approximate location

— 5-foot elevation contour

▭ Approximate location of updated building locations

▭ Approximate property boundary

SOURCES: Google Earth 2020

1 inch = 120 feet

0 60 120 240 Feet

SITE PLAN

48 BED COMMUNITY RTF
VANCOUVER, WASHINGTON

DATE: JUNE 2021 · PROJECT: 73200.011

PBS

FIGURE
2

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Appendix A

Field Explorations

Appendix A: Field Explorations

A1 GENERAL

PBS explored subsurface conditions at the project site by excavating test pits, advancing CPTs, and drilling borings. A total of 17 test pits were excavated to depths of approximately 10 feet bgs on November 14, 2020, and February 20, 2021. Three cone penetration tests (CPTs) were advanced to depths of up to 56 feet bgs on November 18, 2020. Three soil borings were advanced to depths of 31.5 feet bgs on February 15 and 16, 2021. The approximate locations of the explorations are shown on Figure 2, Site Plan. The procedures used to advance the test pits, soil borings, and CPTs, collect samples, and other field techniques are described in detail in the following paragraphs. Unless otherwise noted, all soil sampling and classification procedures followed engineering practices in general accordance with relevant ASTM procedures. "General accordance" means that certain local drilling and excavation and descriptive practices and methodologies have been followed.

A2 BORINGS

A2.1 Drilling

Borings were advanced using a track-mounted CME-850 drill rig provided and operated by Western States Soil Conservation, Inc., of Hubbard, Oregon, using mud rotary and hollow-stem auger drilling techniques. The borings were observed by a member of the PBS geotechnical staff, who maintained a detailed log of the subsurface conditions and materials encountered during the course of the work.

A2.2 Sampling

Disturbed soil samples were taken in the borings at selected depth intervals. The samples were obtained using a standard 2-inch outside diameter, split-spoon sampler following procedures prescribed for the standard penetration test (SPT). Using the SPT, the sampler is driven 18 inches into the soil using a 140-pound hammer dropped 30 inches. The number of blows required to drive the sampler the last 12 inches is defined as the standard penetration resistance (N-value). The N-value provides a measure of the relative density of granular soils such as sands and gravels, and the consistency of cohesive soils such as clays and plastic silts. The disturbed soil samples were examined by a member of the PBS geotechnical staff and then sealed in plastic bags for further examination and physical testing in our laboratory.

A2.3 Boring Logs

The boring logs show the various types of materials that were encountered in the borings and the depths where the materials and/or characteristics of these materials changed, although the changes may be gradual. Where material types and descriptions changed between samples, the contacts were interpreted. The types of samples taken during drilling, along with their sample identification number, are shown to the right of the classification of materials. The N-values and natural water (moisture) contents are shown farther to the right.

A3 TEST PITS

A3.1 Excavation

Test pits were excavated using a Hitachi Zaxis 40U excavator equipped with a 24-inch-wide, toothed bucket provided and operated by Dan J. Fisher Excavating, Inc., of Forest Grove, Oregon. The test pits were observed by a member of the PBS geotechnical staff, who maintained a detailed log of the subsurface conditions and materials encountered during the course of the work.

A3.2 Sampling

Representative disturbed samples were taken at selected depths in the test pits. The disturbed soil samples were examined by a member of the PBS geotechnical staff and sealed in plastic bags for further examination.

A3.3 Test Pit Logs

The test pit logs show the various types of materials that were encountered in the excavations and the depths where the materials and/or characteristics of these materials changed, although the changes may be gradual. Where material types and descriptions changed between samples, the contacts were interpreted. The types of samples taken during excavation, along with their sample identification number, are shown to the right of the classification of materials. The natural water (moisture) contents are shown farther to the right. Measured seepage levels, if observed, are noted in the column to the right.

A4 CONE PENETRATION TESTS (CPT)

A4.1 Field Procedures

Three CPT probes were advanced using a 20-ton truck mounted with a Vertek CPT 10 cm^2 electric seismic piezo cone owned and operated by Geotechnical Explorations, Inc., of Keizer, Oregon. During the test, the instrumented cone is hydraulically pushed into the ground at the rate of about 2 centimeters per second (cm/s), and readings of cone tip resistance, sleeve friction, and pore pressure are digitally recorded every second. As the cone advances, additional cone rods are added such that a “string” of rods continuously advances through the soil. As the test progresses, the CPT operator monitors the cone resistance and its deviation from vertical alignment.

For CPT soundings in which seismic data were collected, conventional CPT testing is temporarily halted at 2-meter intervals to collect seismic data. A seismograph integrated with the CPT is used to record the arrival time of seismic waves generated by striking a steel beam positioned at least 10 feet from the cone rods and coupled to the ground surface by the weight of the beam and operator to prevent the beam from moving when struck.

Each side of the beam is struck several times, and each signal produced by a blow is closely examined for signal and noise content, after which the waveform is selected and the arrival time of the shear wave is determined and recorded. After a complete set of seismic data are recorded, the cone is advanced to the next depth, and the procedure is repeated until the hole is complete.

A4.2 CPT Logs

In accordance with the applicable ASTM standard, the vertical axis is designated for the depth, while the horizontal axis displays the magnitude of the test values recorded. Recorded values include tip and shaft resistance and pore pressure. Final plotting scales are determined after all the tests are complete and take into consideration maximum test values and depths recorded for the project. This information is used to calculate the friction ratio and is correlated to material types, which are presented graphically in a column to the right. The CPT logs are included as Figures A21 through A23. The results of shear wave velocity testing are included on Figure A24.

A5 MATERIAL DESCRIPTION

Initially, samples were classified visually in the field. Consistency, color, relative moisture, degree of plasticity, and other distinguishing characteristics of the soil samples were noted. Afterward, the samples were reexamined in the PBS laboratory, various standard classification tests were conducted, and the field classifications were modified where necessary. The terminology used in the soil classifications and other modifiers are defined in Table A-1, Terminology Used to Describe Soil.

Table A-1
Terminology Used to Describe Soil

Soil Descriptions

Soils exist in mixtures with varying proportions of components. The predominant soil, i.e., greater than 50 percent based on total dry weight, is the primary soil type and is capitalized in our log descriptions (SAND, GRAVEL, SILT, or CLAY). Smaller percentages of other constituents in the soil mixture are indicated by use of modifier words in general accordance with the ASTM D2488-06 Visual-Manual Procedure. “General Accordance” means that certain local and common descriptive practices may have been followed. In accordance with ASTM D2488-06, group symbols (such as GP or CH) are applied on the portion of soil passing the 3-inch (75mm) sieve based on visual examination. The following describes the use of soil names and modifying terms used to describe fine- and coarse-grained soils.

Fine-Grained Soils (50% or greater fines passing 0.075 mm, No. 200 sieve)
The primary soil type, i.e., SILT or CLAY is designated through visual-manual procedures to evaluate soil toughness, dilatency, dry strength, and plasticity. The following outlines the terminology used to describe fine-grained soils, and varies from ASTM D2488 terminology in the use of some common terms.

Primary soil NAME, Symbols, and Adjectives			Plasticity Description	Plasticity Index (PI)
SILT (ML & MH)	**CLAY (CL & CH)**	**ORGANIC SOIL (OL & OH)**		
SILT		Organic SILT	Non-plastic	0 – 3
SILT		Organic SILT	Low plasticity	4 – 10
SILT/Elastic SILT	Lean CLAY	Organic SILT/ Organic CLAY	Medium Plasticity	10 – 20
Elastic SILT	Lean/Fat CLAY	Organic CLAY	High Plasticity	20 – 40
Elastic SILT	Fat CLAY	Organic CLAY	Very Plastic	>40

Modifying terms describing secondary constituents, estimated to 5 percent increments, are applied as follows:

Description	% Composition	
With Sand	% Sand ≥ % Gravel	15% to 25% plus No. 200
With Gravel	% Sand < % Gravel	
Sandy	% Sand ≥ % Gravel	≤30% to 50% plus No. 200
Gravelly	% Sand < % Gravel	

Borderline Symbols, for example CH/MH, are used when soils are not distinctly in one category or when variable soil units contain more than one soil type. **Dual Symbols,** for example CL-ML, are used when two symbols are required in accordance with ASTM D2488.

Soil Consistency terms are applied to fine-grained, plastic soils (i.e., PI ≥ 7). Descriptive terms are based on direct measure or correlation to the Standard Penetration Test N-value as determined by ASTM D1586-84, as follows. SILT soils with low to non-plastic behavior (i.e., PI < 7) may be classified using relative density.

Consistency Term	SPT N-value	Unconfined Compressive Strength	
		tsf	kPa
Very soft	Less than 2	Less than 0.25	Less than 24
Soft	2 – 4	0.25 – 0.5	24 – 48
Medium stiff	5 – 8	0.5 – 1.0	48 – 96
Stiff	9 – 15	1.0 – 2.0	96 – 192
Very stiff	16 – 30	2.0 – 4.0	192 – 383
Hard	Over 30	Over 4.0	Over 383

Soil Descriptions

Coarse - Grained Soils (less than 50% fines)

Coarse-grained soil descriptions, i.e., SAND or GRAVEL, are based on the portion of materials passing a 3-inch (75mm) sieve. Coarse-grained soil group symbols are applied in accordance with ASTM D2488-06 based on the degree of grading, or distribution of grain sizes of the soil. For example, well-graded sand containing a wide range of grain sizes is designated SW; poorly graded gravel, GP, contains high percentages of only certain grain sizes. Terms applied to grain sizes follow.

Material NAME	Particle Diameter	
	Inches	Millimeters
SAND (SW or SP)	0.003 – 0.19	0.075 – 4.8
GRAVEL (GW or GP)	0.19 – 3	4.8 – 75
Additional Constituents:		
Cobble	3 – 12	75 – 300
Boulder	12 – 120	300 – 3050

The primary soil type is capitalized, and the fines content in the soil are described as indicated by the following examples. Percentages are based on estimating amounts of fines, sand, and gravel to the nearest 5 percent. Other soil mixtures will have similar descriptive names.

Example: Coarse-Grained Soil Descriptions with Fines

>5% to < 15% fines (Dual Symbols)	≥15% to < 50% fines
Well graded GRAVEL with silt: GW-GM	Silty GRAVEL: GM
Poorly graded SAND with clay: SP-SC	Silty SAND: SM

Additional descriptive terminology applied to coarse-grained soils follow.

Example: Coarse-Grained Soil Descriptions with Other Coarse-Grained Constituents

Coarse-Grained Soil Containing Secondary Constituents	
With sand or with gravel	≥ 15% sand or gravel
With cobbles; with boulders	Any amount of cobbles or boulders.

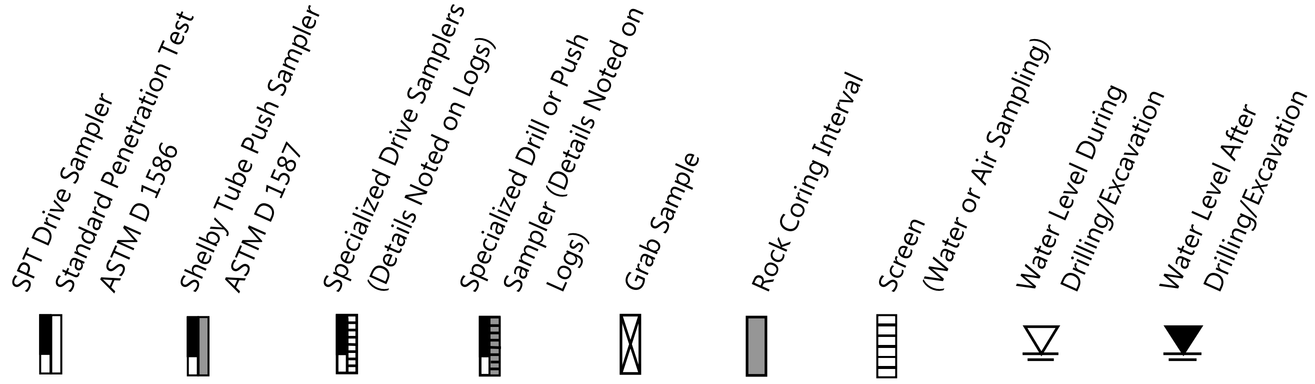
Cobble and boulder deposits may include a description of the matrix soils, as defined above.

Relative Density terms are applied to granular, non-plastic soils based on direct measure or correlation to the Standard Penetration Test N-value as determined by ASTM D1586-84.

Relative Density Term	SPT N-value
Very loose	0 – 4
Loose	5 – 10
Medium dense	11 – 30
Dense	31 – 50
Very dense	> 50

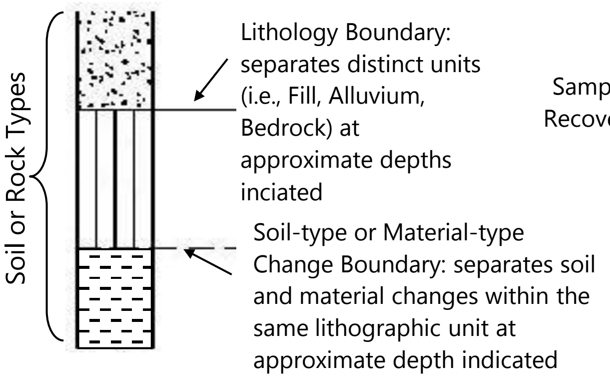
Table A-2
Key To Test Pit and Boring Log Symbols

SAMPLING DESCRIPTIONS

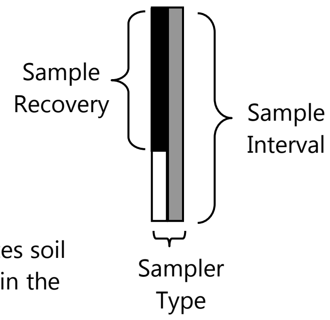


LOG GRAPHICS

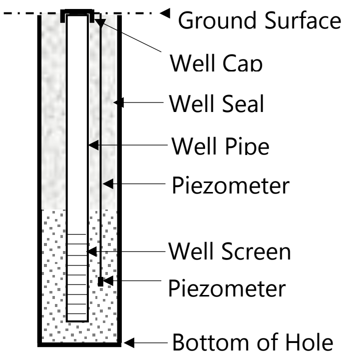
Soil and Rock



Sampling Symbols



Instrumentation Detail



Geotechnical Testing Acronym Explanations

PP	Pocket Penetrometer	HYD	Hydrometer Gradation
TOR	Torvane	SIEV	Sieve Gradation
DCP	Dynamic Cone Penetrometer	DS	Direct Shear
ATT	Atterberg Limits	DD	Dry Density
PL	Plasticity Limit	CBR	California Bearing Ratio
LL	Liquid Limit	RES	Resilient Modulus
PI	Plasticity Index	VS	Vane Shear
P200	Percent Passing US Standard No. 200 Sieve	bgs	Below ground surface
OC	Organic Content	MSL	Mean Sea Level
CON	Consolidation	HCL	Hydrochloric Acid
UC	Unconfined Compressive Strength		

Details of soil and rock classification systems are available on request.

48 BED COMMUNITY RTF
VANCOUVER, WASHINGTON

PBS PROJECT NUMBER:
73200.011

BORING B-1

APPROX. BORING B-1 LOCATION:
(See Site Plan)
Lat: 45.73778 Long: -122.61750

NOTE: Lines representing the interface between soil/rock units of differing description are approximate only, inferred where between samples, and may indicate gradual transition.

▲ UNCORRECTED N-VALUE
✦ DYNAMIC CONE PENETROMETER
● MOISTURE CONTENT %
▥ RQD% ▨ CORE REC%

Surface Conditions: Topsoil

DEPTH FEET	MATERIAL DESCRIPTION	DEPTH	TESTING	SAMPLE TYPE SAMPLE ID	UNCORRECTED N-VALUE	INSTALLATION AND COMMENTS
0.0	TOPSOIL/TILL ZONE	0.0				Flush-mount Monument
	Loose, brown to gray with iron oxide staining, silty SAND (SM); low plasticity; fine sand; moist	2.0		S-1		∇ 02/15/21
5.0			P200	S-2	6	HSA to 5 feet bgs; switched to mud rotary P200 = 41%
				S-3	6	2-inch-diameter PVC
	becomes medium dense; fine to medium sand; wet					
10.0				S-4	14	3/8-inch Bentonite Chips
15.0	becomes fine sand			S-5	16	
						12/20 Sand
20.0				S-6	15	
						0.010-inch Slotted PVC
25.0	Medium stiff, gray, sandy SILT (ML); non-plastic; fine sand; wet	25.0	P200	S-7	7	P200 = 66%
30.0	becomes stiff, dark gray			S-8	13	
	Final depth 31.5 feet bgs; piezometer installed to 30.0 feet bgs.	31.5				
35.0						

DRILLING METHOD: Hollow-Stem Auger/Mud Rotary
DRILLED BY: Western States Soil Conservation, Inc.
LOGGED BY: F. Jarman

BIT DIAMETER: 6¼ inches; 4 7/8 inches
HAMMER EFFICIENCY PERCENT: 88.3
LOGGING COMPLETED: 2/15/2021

FIGURE A1

48 BED COMMUNITY RTF
VANCOUVER, WASHINGTON

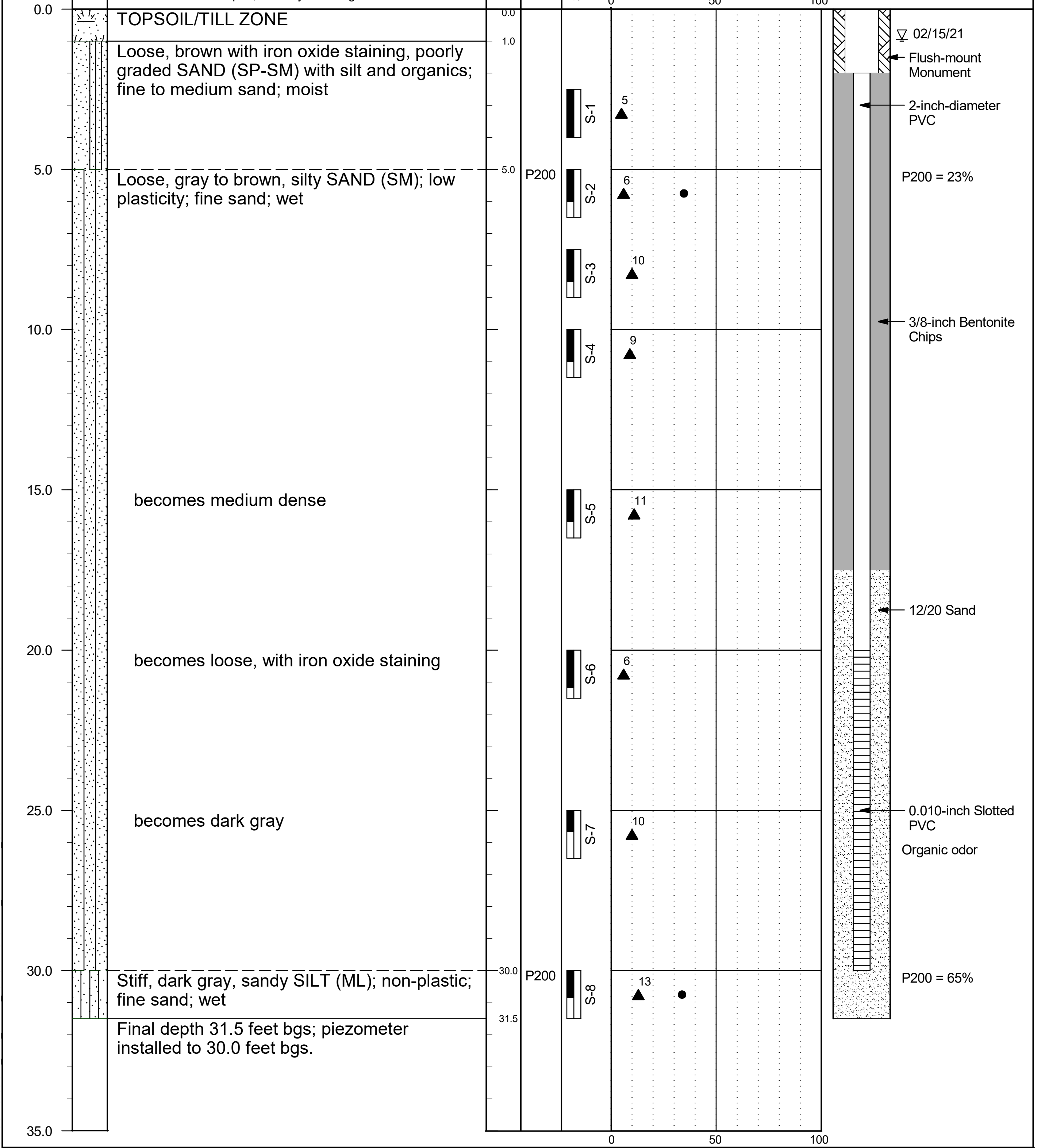
PBS PROJECT NUMBER:
73200.011

BORING B-2

APPROX. BORING B-2 LOCATION:
(See Site Plan)
Lat: 45.73722 Long: -122.61722

DEPTH FEET	MATERIAL DESCRIPTION	DEPTH	TESTING	SAMPLE TYPE SAMPLE ID	UNCORRECTED N-VALUE	INSTALLATION AND COMMENTS
0.0	TOPSOIL/TILL ZONE	0.0				Surface Conditions: Topsoil
	Loose, brown with iron oxide staining, poorly graded SAND (SP-SM) with silt and organics; fine to medium sand; moist	1.0		S-1	5	
5.0	Loose, gray to brown, silty SAND (SM); low plasticity; fine sand; wet	5.0	P200	S-2	6	P200 = 23%
				S-3	10	
10.0				S-4	9	
15.0	becomes medium dense			S-5	11	
20.0	becomes loose, with iron oxide staining			S-6	6	
25.0	becomes dark gray			S-7	10	Organic odor
30.0	Stiff, dark gray, sandy SILT (ML); non-plastic; fine sand; wet	30.0	P200	S-8	13	P200 = 65%
	Final depth 31.5 feet bgs; piezometer installed to 30.0 feet bgs.	31.5				
35.0						

NOTE: Lines representing the interface between soil/rock units of differing description are approximate only, inferred where between samples, and may indicate gradual transition.



DRILLING METHOD: Mud Rotary
DRILLED BY: Western States Soil Conservation, Inc.
LOGGED BY: F. Jarman

BIT DIAMETER: 4 7/8 inches
HAMMER EFFICIENCY PERCENT: 88.3
LOGGING COMPLETED: 2/15/2021

FIGURE A2
Page 1 of 1

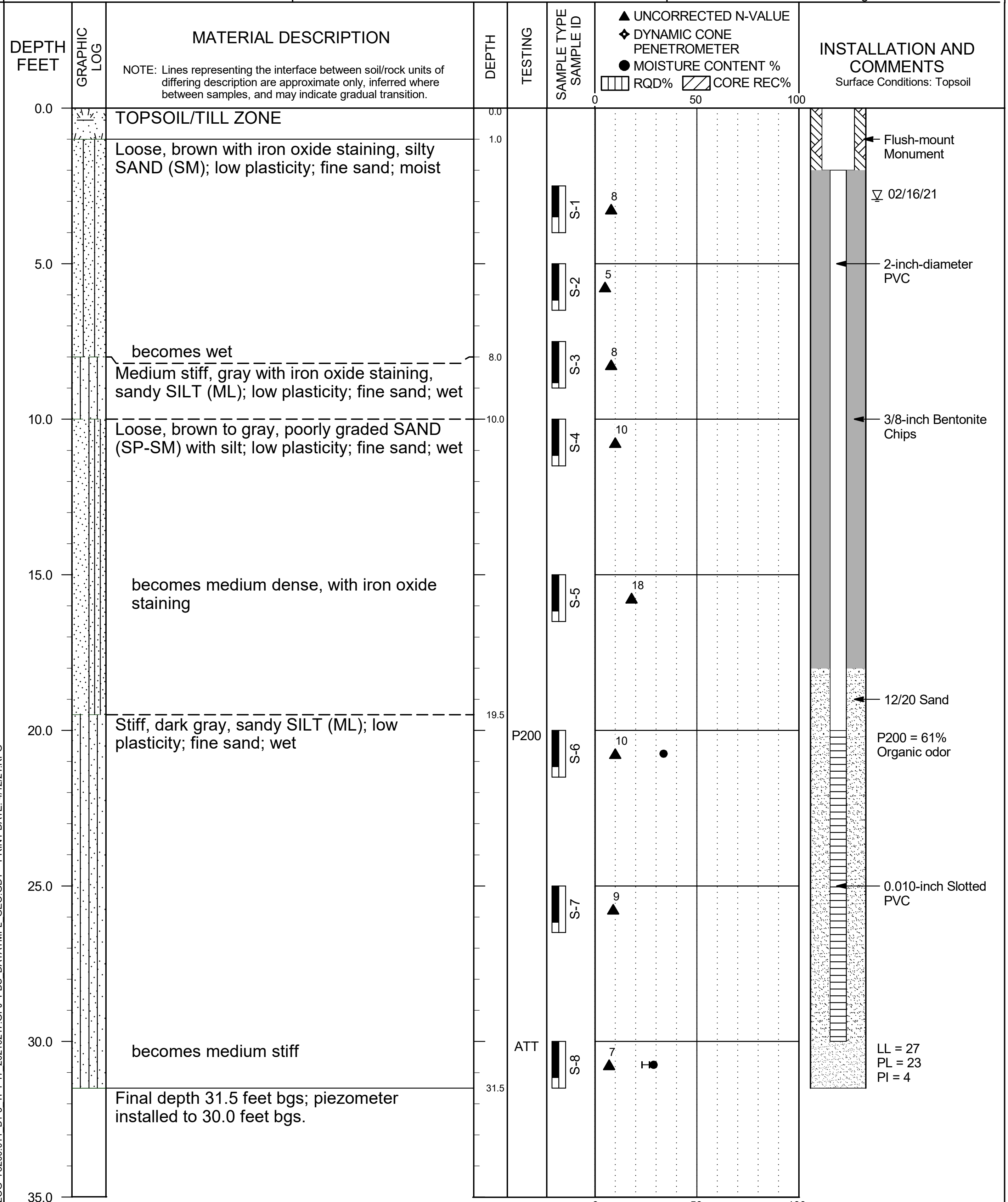
BORING LOG 73200.011 B1-3 TP1-17 20210217.GPJ PBS DATATMPL GEO.GDT PRINT DATE: 4/12/21:RPG

48 BED COMMUNITY RTF
VANCOUVER, WASHINGTON

BORING B-3

PBS PROJECT NUMBER:
73200.011

APPROX. BORING B-3 LOCATION:
(See Site Plan)
Lat: 45.73667 Long: -122.61556



NOTE: Lines representing the interface between soil/rock units of differing description are approximate only, inferred where between samples, and may indicate gradual transition.

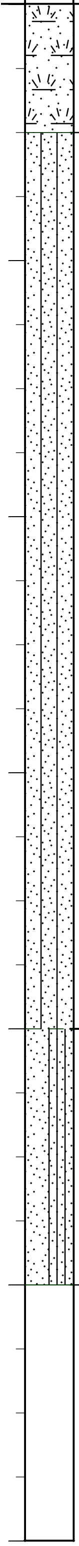
DEPTH	MATERIAL DESCRIPTION	TESTING	SAMPLE ID	UNCORRECTED N-VALUE
0.0	TOPSOIL/TILL ZONE			
1.0	Loose, brown with iron oxide staining, silty SAND (SM); low plasticity; fine sand; moist		S-1	8
			S-2	5
8.0	becomes wet		S-3	8
8.0	Medium stiff, gray with iron oxide staining, sandy SILT (ML); low plasticity; fine sand; wet			
10.0	Loose, brown to gray, poorly graded SAND (SP-SM) with silt; low plasticity; fine sand; wet		S-4	10
	becomes medium dense, with iron oxide staining		S-5	18
19.5	Stiff, dark gray, sandy SILT (ML); low plasticity; fine sand; wet	P200	S-6	10
			S-7	9
	becomes medium stiff	ATT	S-8	7
31.5	Final depth 31.5 feet bgs; piezometer installed to 30.0 feet bgs.			

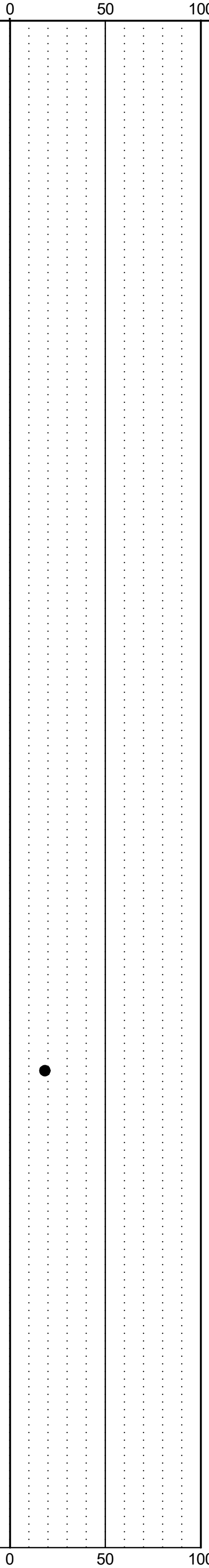
DRILLING METHOD: Mud Rotary
DRILLED BY: Western States Soil Conservation, Inc.
LOGGED BY: F. Jarman

BIT DIAMETER: 4 7/8 inches
HAMMER EFFICIENCY PERCENT: 88.3
LOGGING COMPLETED: 2/16/2021

PBS	48 BED COMMUNITY RTF VANCOUVER, WASHINGTON	**TEST PIT TP-1**
	PBS PROJECT NUMBER: 73200.011	APPROX. TEST PIT TP-1 LOCATION: (See Site Plan) Lat: 45.7377 Long: -122.6182

DEPTH FEET	MATERIAL DESCRIPTION Lines representing the interface between soil/rock units of differing description are approximate only, inferred where between samples, and may indicate gradual transition.	DEPTH	TESTING	SAMPLE TYPE SAMPLE ID	◆ DYNAMIC CONE PENETROMETER ☒ STATIC PENETROMETER ● MOISTURE CONTENT % 0 50 100	COMMENTS Surface Conditions: Topsoil
0.0	TOPSOIL/TILL ZONE: Brown, sandy SILT (ML) with organics; low plasticity; fine sand; moist	0.0				
	Brown to gray, silty SAND (SM); low plasticity; fine sand; moist	1.0				
2.0						
4.0				☒ S-1		
6.0				☒ S-2		
8.0	Tan, poorly graded SAND (SP-SM) with silt; low plasticity; fine sand; moist	8.0	P200	☒ S-3	●	P200 = 12%
10.0	Final depth 10.0 feet bgs; test pit backfilled with excavated material to existing ground surface. Groundwater not encountered at time of exploration.	10.0				
12.0					0 50 100	





LOGGED BY: F. Jarman
COMPLETED: 11/14/2020

EXCAVATED BY: Dan J. Fischer Excavating, Inc.
EXCAVATION METHOD: Mini Excavator with 24" Bucket

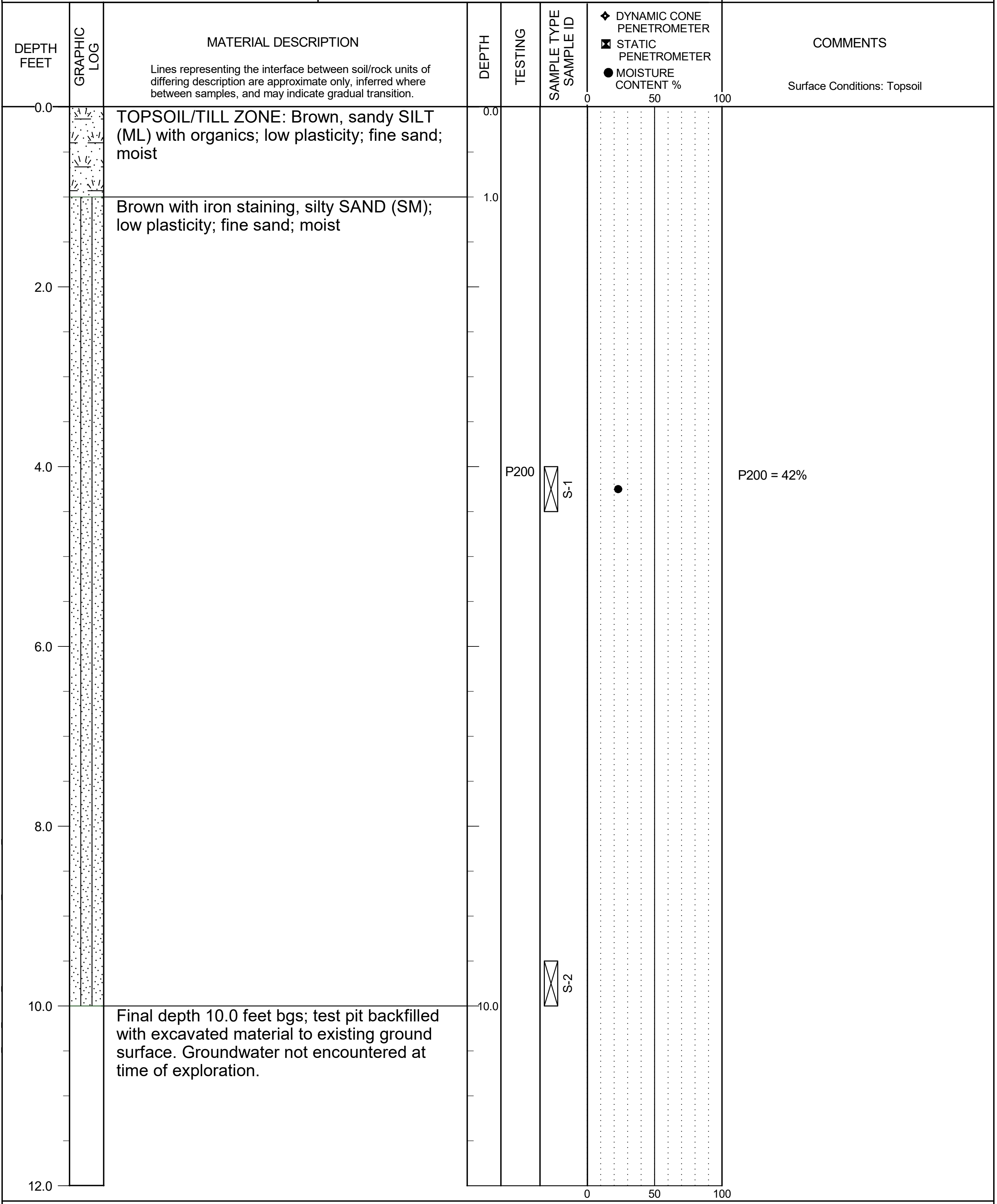
FIGURE A4

48 BED COMMUNITY RTF
VANCOUVER, WASHINGTON

TEST PIT TP-2

PBS PROJECT NUMBER:
73200.011

APPROX. TEST PIT TP-2 LOCATION:
(See Site Plan)
Lat: 45.7378 Long: -122.6187



MATERIAL DESCRIPTION

Lines representing the interface between soil/rock units of differing description are approximate only, inferred where between samples, and may indicate gradual transition.

COMMENTS

Surface Conditions: Topsoil

Depth	Material Description	Testing	Sample ID	Comments
0.0	TOPSOIL/TILL ZONE: Brown, sandy SILT (ML) with organics; low plasticity; fine sand; moist			
1.0	Brown with iron staining, silty SAND (SM); low plasticity; fine sand; moist			
4.0		P200	S-1	P200 = 42%
			S-2	
10.0	Final depth 10.0 feet bgs; test pit backfilled with excavated material to existing ground surface. Groundwater not encountered at time of exploration.			

LOGGED BY: F. Jarman
COMPLETED: 11/14/2020

EXCAVATED BY: Dan J. Fischer Excavating, Inc.
EXCAVATION METHOD: Mini Excavator with 24" Bucket

FIGURE A5
Page 1 of 1

PBS

48 BED COMMUNITY RTF
VANCOUVER, WASHINGTON

TEST PIT TP-3

PBS PROJECT NUMBER:
73200.011

APPROX. TEST PIT TP-3 LOCATION:
(See Site Plan)
Lat: 45.7387 Long: -122.6196

DEPTH FEET	MATERIAL DESCRIPTION Lines representing the interface between soil/rock units of differing description are approximate only, inferred where between samples, and may indicate gradual transition.	DEPTH	TESTING	SAMPLE TYPE SAMPLE ID	◆ DYNAMIC CONE PENETROMETER ☒ STATIC PENETROMETER ● MOISTURE CONTENT % 0 50 100	COMMENTS Surface Conditions: Topsoil
0.0	TOPSOIL/TILL ZONE: Brown, sandy SILT (ML) with organics; low plasticity; fine sand; moist	0.0				
1.0	Brown, sandy SILT (ML); low plasticity; fine sand; moist	1.0				
4.0	Brown with iron staining, silty SAND (SM); low plasticity; fine sand; moist	4.0	P200	☒ S-1	●	P200 = 36%
10.0	Final depth 10.0 feet bgs; test pit backfilled with excavated material to existing ground surface. Groundwater not encountered at time of exploration.	10.0				
12.0					0 50 100	

LOGGED BY: F. Jarman
COMPLETED: 11/14/2020

EXCAVATED BY: Dan J. Fischer Excavating, Inc.
EXCAVATION METHOD: Mini Excavator with 24" Bucket

FIGURE A6
Page 1 of 1

TEST PIT LOG - 1 PER PAGE 73200.011 B1-3 TP1-17 20210217.GPJ PBS DATATMPL GEO.GDT PRINT DATE: 4/12/21:RPG

48 BED COMMUNITY RTF
VANCOUVER, WASHINGTON

TEST PIT TP-4

PBS PROJECT NUMBER:
73200.011

APPROX. TEST PIT TP-4 LOCATION:
(See Site Plan)

Lat: Long:

DEPTH FEET	MATERIAL DESCRIPTION Lines representing the interface between soil/rock units of differing description are approximate only, inferred where between samples, and may indicate gradual transition.	DEPTH	TESTING	SAMPLE TYPE SAMPLE ID	◆ DYNAMIC CONE PENETROMETER ☒ STATIC PENETROMETER ● MOISTURE CONTENT % 0 50 100	COMMENTS Surface Conditions: Topsoil
0.0	TOPSOIL/TILL ZONE: Brown, sandy SILT (ML) with organics; low plasticity; fine sand; moist	0.0				
	Brown, silty SAND (SM); low plasticity; fine sand; moist	1.0				
2.0						
4.0				☒ S-1		
6.0						
	Gray, poorly graded SAND (SP-SM) with silt; low plasticity; fine sand; moist	7.0		☒ S-2		
8.0						
10.0	Final depth 10.0 feet bgs; test pit backfilled with excavated material to existing ground surface. Groundwater not encountered at time of exploration.	10.0				
12.0					0 50 100	

LOGGED BY: F. Jarman
COMPLETED: 11/14/2020

EXCAVATED BY: Dan J. Fischer Excavating, Inc.
EXCAVATION METHOD: Mini Excavator with 24" Bucket

FIGURE A7

48 BED COMMUNITY RTF
VANCOUVER, WASHINGTON

TEST PIT TP-5

PBS PROJECT NUMBER:
73200.011

APPROX. TEST PIT TP-5 LOCATION:
(See Site Plan)
Lat: 45.7371 Long: -122.6196

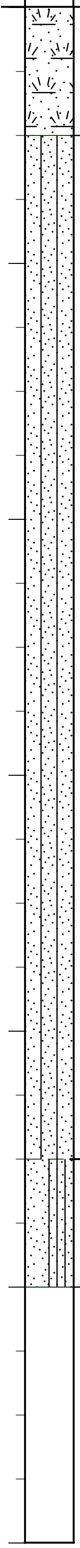
MATERIAL DESCRIPTION

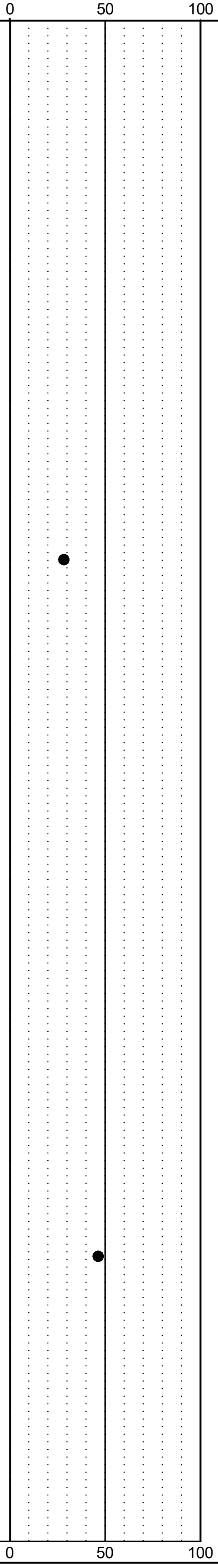
Lines representing the interface between soil/rock units of differing description are approximate only, inferred where between samples, and may indicate gradual transition.

❖ DYNAMIC CONE PENETROMETER
☒ STATIC PENETROMETER
● MOISTURE CONTENT %

Surface Conditions: Topsoil

DEPTH FEET	MATERIAL DESCRIPTION	DEPTH	TESTING	SAMPLE TYPE SAMPLE ID	COMMENTS
0.0	TOPSOIL/TILL ZONE: Brown, silty SAND (SM) with organics; low plasticity; fine sand; moist	0.0			
	Brown with iron staining, silty SAND (SM); low plasticity; fine sand; moist	1.0			
4.0				☒ S-1	
	becomes tan			☒ S-2	
	Gray, poorly graded SAND (SP-SM) with silt; low plasticity; fine sand; micaceous; moist	9.0			Infiltration testing completed at 10 feet bgs
			P200	☒ S-3	P200 = 11%
10.0	Final depth 10.0 feet bgs; test pit backfilled with excavated material to existing ground surface. Groundwater not encountered at time of exploration.	10.0			





LOGGED BY: F. Jarman
COMPLETED: 11/14/2020

EXCAVATED BY: Dan J. Fischer Excavating, Inc.
EXCAVATION METHOD: Mini Excavator with 24" Bucket

FIGURE A8
Page 1 of 1

48 BED COMMUNITY RTF
VANCOUVER, WASHINGTON

PBS PROJECT NUMBER:
73200.011

TEST PIT TP-6

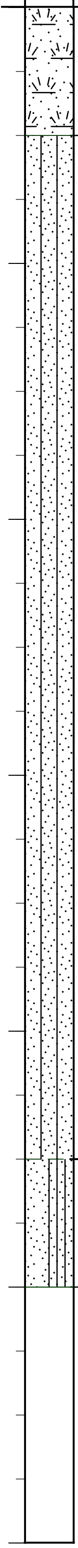
APPROX. TEST PIT TP-6 LOCATION:
(See Site Plan)
Lat: 45.7371 Long: -122.6203

Surface Conditions: Topsoil

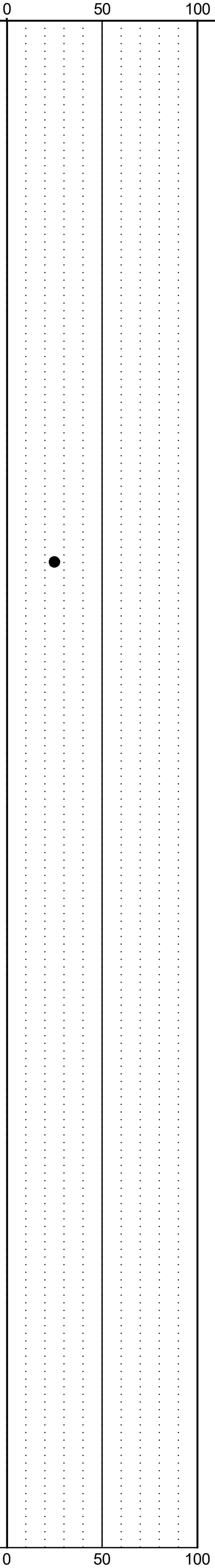
MATERIAL DESCRIPTION

Lines representing the interface between soil/rock units of differing description are approximate only, inferred where between samples, and may indicate gradual transition.

Legend: ✦ DYNAMIC CONE PENETROMETER; ☒ STATIC PENETROMETER; ● MOISTURE CONTENT %



DEPTH FEET	MATERIAL DESCRIPTION	DEPTH	TESTING	SAMPLE TYPE SAMPLE ID	COMMENTS
0.0	TOPSOIL/TILL ZONE: Brown, sandy SILT (ML) with organics; low plasticity; fine sand; moist	0.0			
1.0	Brown to gray with iron staining, silty SAND (SM); low plasticity; fine sand; moist	1.0			
4.0				S-1	
6.0	becomes tan			S-2	
9.0	Gray, poorly graded SAND (SP-SM) with silt; low plasticity; fine sand; micaceous; moist	9.0		S-3	
10.0	Final depth 10.0 feet bgs; test pit backfilled with excavated material to existing ground surface. Groundwater not encountered at time of exploration.	10.0			



LOGGED BY: F. Jarman
COMPLETED: 11/14/2020

EXCAVATED BY: Dan J. Fischer Excavating, Inc.
EXCAVATION METHOD: Mini Excavator with 24" Bucket

FIGURE A9

48 BED COMMUNITY RTF
VANCOUVER, WASHINGTON

TEST PIT TP-7

PBS PROJECT NUMBER:
73200.011

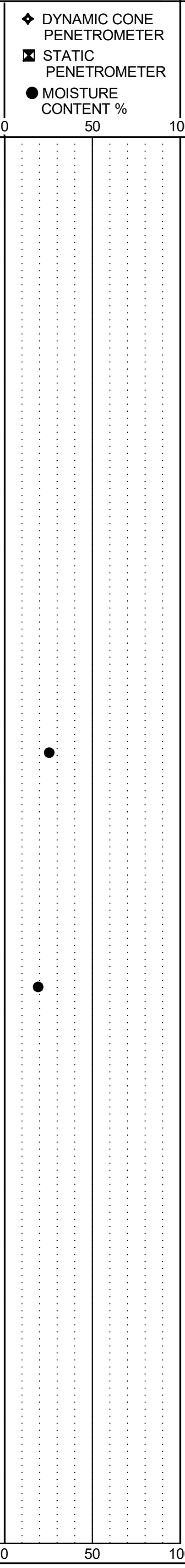
APPROX. TEST PIT TP-7 LOCATION:
(See Site Plan)
Lat: 45.7376 Long: -122.6202

MATERIAL DESCRIPTION

Lines representing the interface between soil/rock units of differing description are approximate only, inferred where between samples, and may indicate gradual transition.

Surface Conditions: Topsoil

DEPTH FEET	MATERIAL DESCRIPTION	DEPTH	TESTING	SAMPLE TYPE SAMPLE ID	COMMENTS
0.0	TOPSOIL/TILL ZONE: Brown, silty SAND (SM) with organics; low plasticity; fine sand; moist	0.0			
1.0	Brown with iron staining, silty SAND (SM); low plasticity; fine sand; moist	1.0			
4.0				S-1	
4.5					Infiltration testing completed at 5 feet bgs
5.0			P200	S-2	P200 = 31%
7.0	becomes tan		P200	S-3	P200 = 28%
10.0	Final depth 10.0 feet bgs; test pit backfilled with excavated material to existing ground surface. Groundwater not encountered at time of exploration.	10.0			



LOGGED BY: F. Jarman
COMPLETED: 11/14/2020

EXCAVATED BY: Dan J. Fischer Excavating, Inc.
EXCAVATION METHOD: Mini Excavator with 24" Bucket

FIGURE A10
Page 1 of 1

TEST PIT LOG - 1 PER PAGE 73200.011 B1-3 TP1-17 20210217.GPJ PBS DATATMPL GEO.GDT PRINT DATE: 4/12/21:RPG

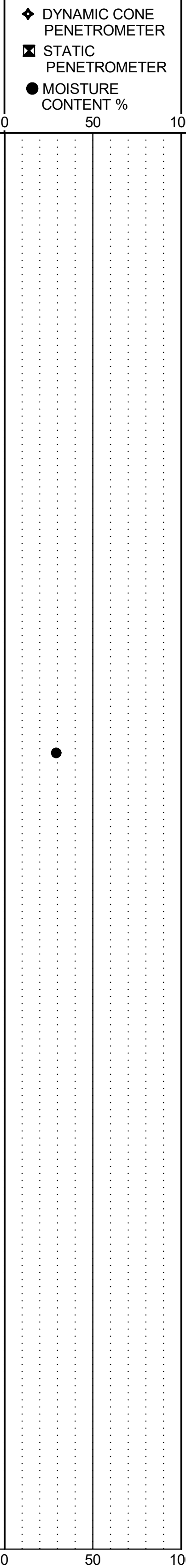
48 BED COMMUNITY RTF
VANCOUVER, WASHINGTON

PBS PROJECT NUMBER:
73200.011

TEST PIT TP-8

APPROX. TEST PIT TP-8 LOCATION:
(See Site Plan)
Lat: 45.7376 Long: -122.6189

DEPTH FEET	MATERIAL DESCRIPTION Lines representing the interface between soil/rock units of differing description are approximate only, inferred where between samples, and may indicate gradual transition.	DEPTH	TESTING	SAMPLE TYPE SAMPLE ID	COMMENTS Surface Conditions: Topsoil
0.0	TOPSOIL/TILL ZONE: Brown, silty SAND (SM) with organics; low plasticity; fine sand; moist	0.0			
	Brown to gray with iron staining, silty SAND (SM); low plasticity; fine sand; moist	1.0			
2.0					
4.0				S-1	Infiltration testing completed at 5 feet bgs
			P200	S-2	P200 = 36%
6.0					
8.0	Gray, poorly graded SAND (SP-SM) with silt; low plasticity; fine sand; micaceous; moist	8.0		S-3	
10.0	Final depth 10.0 feet bgs; test pit backfilled with excavated material to existing ground surface. Groundwater not encountered at time of exploration.	10.0			
12.0					



LOGGED BY: F. Jarman
COMPLETED: 11/14/2020

EXCAVATED BY: Dan J. Fischer Excavating, Inc.
EXCAVATION METHOD: Mini Excavator with 24" Bucket

FIGURE A11

PBS

48 BED COMMUNITY RTF
VANCOUVER, WASHINGTON

PBS PROJECT NUMBER:
73200.011

TEST PIT TP-9

APPROX. TEST PIT TP-9 LOCATION:
(See Site Plan)
Lat: 45.73815 Long: -122.61724

DEPTH FEET	MATERIAL DESCRIPTION Lines representing the interface between soil/rock units of differing description are approximate only, inferred where between samples, and may indicate gradual transition.	DEPTH	TESTING	SAMPLE TYPE SAMPLE ID	◆ DYNAMIC CONE PENETROMETER ☒ STATIC PENETROMETER ● MOISTURE CONTENT % 0 50 100	COMMENTS Surface Conditions: Grass
0.0	TOPSOIL/TILL ZONE	0.0				
	Brown, silty SAND (SM); non-plastic; fine to coarse sand; moist	1.0		☒ S-1		
2.0						
	becomes wet					▽ 02/20/21 Seepage at 2.5 feet bgs
	becomes light brown and gray					
4.0						
	becomes tan; fine to medium sand					
6.0						
			P200	☒ S-2		P200 = 48%
8.0						
	Tan, poorly graded SAND (SP-SM) with silt; non-plastic; fine sand; wet	8.5				
10.0	Final depth 10.0 feet bgs; test pit backfilled with excavated material to existing ground surface.	10.0				
12.0					0 50 100	

LOGGED BY: D. Eibert
COMPLETED: 2/20/2021

EXCAVATED BY: Dan J. Fischer Excavating, Inc.
EXCAVATION METHOD: Mini Excavator with 24" Bucket

FIGURE A12
Page 1 of 1

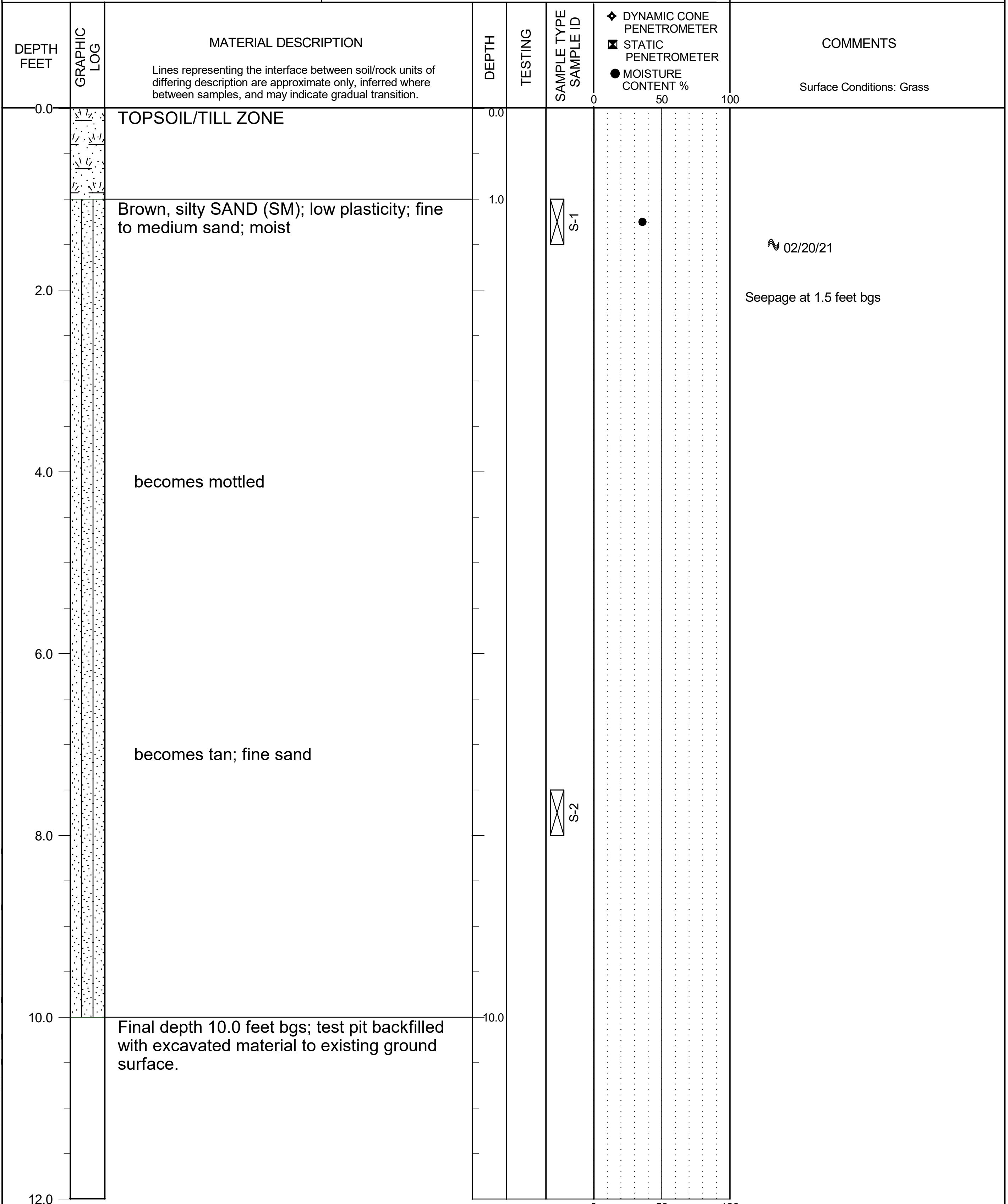
48 BED COMMUNITY RTF
VANCOUVER, WASHINGTON

PBS PROJECT NUMBER:
73200.011

TEST PIT TP-10

APPROX. TEST PIT TP-10 LOCATION:
(See Site Plan)

Lat: 45.73802 Long: -122.61696



MATERIAL DESCRIPTION

Lines representing the interface between soil/rock units of differing description are approximate only, inferred where between samples, and may indicate gradual transition.

COMMENTS

Surface Conditions: Grass

DEPTH FEET	MATERIAL DESCRIPTION	SAMPLE TYPE SAMPLE ID	COMMENTS
0.0	TOPSOIL/TILL ZONE		
1.0	Brown, silty SAND (SM); low plasticity; fine to medium sand; moist	S-1	02/20/21 Seepage at 1.5 feet bgs
	becomes mottled		
	becomes tan; fine sand	S-2	
10.0	Final depth 10.0 feet bgs; test pit backfilled with excavated material to existing ground surface.		

LOGGED BY: D. Eibert
COMPLETED: 2/20/2021

EXCAVATED BY: Dan J. Fischer Excavating, Inc.
EXCAVATION METHOD: Mini Excavator with 24" Bucket

FIGURE A13

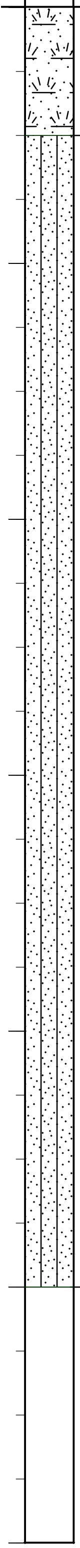
48 BED COMMUNITY RTF
VANCOUVER, WASHINGTON

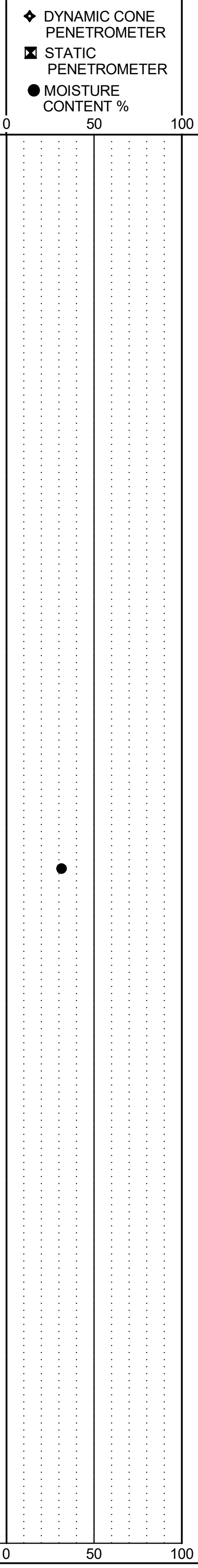
PBS PROJECT NUMBER:
73200.011

TEST PIT TP-11

APPROX. TEST PIT TP-11 LOCATION:
(See Site Plan)
Lat: 45.73800 Long: -122.61668

DEPTH FEET	MATERIAL DESCRIPTION Lines representing the interface between soil/rock units of differing description are approximate only, inferred where between samples, and may indicate gradual transition.	DEPTH	TESTING	SAMPLE TYPE SAMPLE ID	COMMENTS Surface Conditions: Grass
0.0	TOPSOIL/TILL ZONE	0.0			∿ 02/20/21
1.0	Brown, silty SAND (SM); low plasticity; fine to medium sand; moist	1.0			Seepage at 0.5 foot bgs Caving 0 to 10 feet bgs
2.0	becomes wet				
3.0				☒ S-1	
6.0	becomes gray to brown			☒ S-2	
10.0	Final depth 10.0 feet bgs; test pit backfilled with excavated material to existing ground surface.	10.0			





LOGGED BY: D. Eibert
COMPLETED: 2/20/2021

EXCAVATED BY: Dan J. Fischer Excavating, Inc.
EXCAVATION METHOD: Mini Excavator with 24" Bucket

FIGURE A14
Page 1 of 1

48 BED COMMUNITY RTF
VANCOUVER, WASHINGTON

PBS PROJECT NUMBER:
73200.011

TEST PIT TP-12

APPROX. TEST PIT TP-12 LOCATION:
(See Site Plan)
Lat: 45.73752 Long: -122.61720

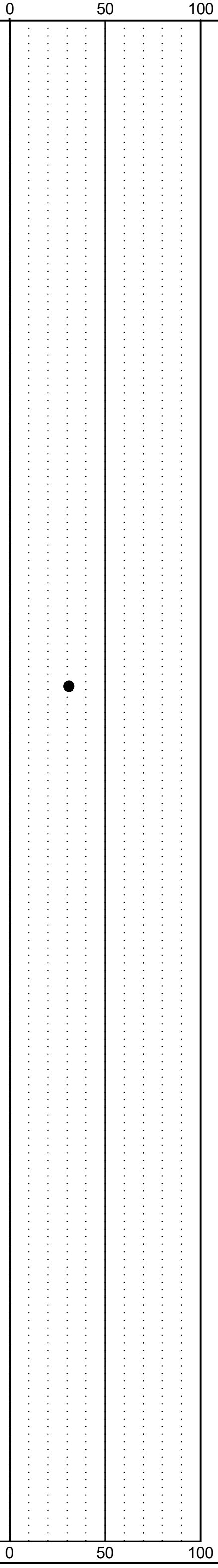
MATERIAL DESCRIPTION

Lines representing the interface between soil/rock units of differing description are approximate only, inferred where between samples, and may indicate gradual transition.

◆ DYNAMIC CONE PENETROMETER
☒ STATIC PENETROMETER
● MOISTURE CONTENT %

Surface Conditions: Grass

DEPTH FEET	MATERIAL DESCRIPTION	SAMPLE TYPE SAMPLE ID	COMMENTS
0.0	TOPSOIL/TILL ZONE		
	roots to 2 feet bgs		≈ 02/20/21
2.0	Light brown, silty SAND (SM); low plasticity; fine to medium sand; wet		Seepage at 1.5 feet bgs
4.0	sparse gravel and single cobble encountered at 4 feet bgs		
	becomes brown to gray	☒ S-1	
		☒ S-2	Caving below 7 feet bgs
10.0	Final depth 10.0 feet bgs; test pit backfilled with excavated material to existing ground surface.		



LOGGED BY: D. Eibert
COMPLETED: 2/20/2021

EXCAVATED BY: Dan J. Fischer Excavating, Inc.
EXCAVATION METHOD: Mini Excavator with 24" Bucket

FIGURE A15

PBS

48 BED COMMUNITY RTF
VANCOUVER, WASHINGTON

PBS PROJECT NUMBER:
73200.011

TEST PIT TP-13

APPROX. TEST PIT TP-13 LOCATION:
(See Site Plan)

Lat: 45.73770 Long: -122.61685

◆ DYNAMIC CONE PENETROMETER
☒ STATIC PENETROMETER
● MOISTURE CONTENT %
0 50 100

Surface Conditions: Grass

DEPTH FEET	MATERIAL DESCRIPTION Lines representing the interface between soil/rock units of differing description are approximate only, inferred where between samples, and may indicate gradual transition.	DEPTH	TESTING	SAMPLE TYPE SAMPLE ID	COMMENTS
0.0	TOPSOIL/TILL ZONE	0.0			
	Brown, silty SAND (SM); low plasticity; fine to medium sand; moist becomes wet	1.0			∿ 02/20/21 Seepage at 1.25 feet bgs
2.0					
	Brown, silty SAND (SM) with gravel; non-plastic; fine to medium sand; fine, rounded gravel; wet	2.5		☒ S-1	
4.0					
	Brown to gray, silty SAND (SM); non-plastic; fine to medium sand; wet	4.5			
6.0					
8.0			P200	☒ S-2	P200 = 30%
10.0	Final depth 10.0 feet bgs; test pit backfilled with excavated material to existing ground surface.	10.0			
12.0					

0 50 100

LOGGED BY: D. Eibert
COMPLETED: 2/20/2021

EXCAVATED BY: Dan J. Fischer Excavating, Inc.
EXCAVATION METHOD: Mini Excavator with 24" Bucket

FIGURE A16

PBS

48 BED COMMUNITY RTF
VANCOUVER, WASHINGTON

PBS PROJECT NUMBER:
73200.011

TEST PIT TP-14

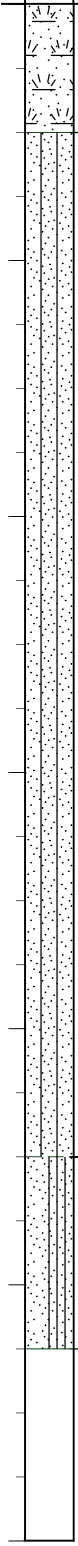
APPROX. TEST PIT TP-14 LOCATION:
(See Site Plan)
Lat: 45.73750 Long: -122.61648

Surface Conditions: Grass

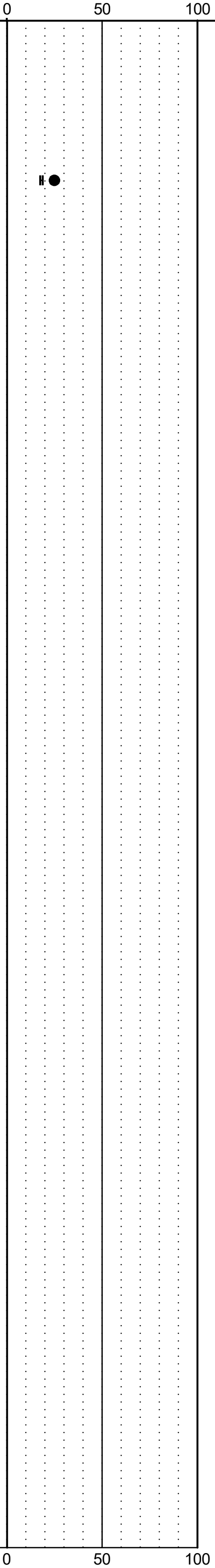
MATERIAL DESCRIPTION

Lines representing the interface between soil/rock units of differing description are approximate only, inferred where between samples, and may indicate gradual transition.

◆ DYNAMIC CONE PENETROMETER
☒ STATIC PENETROMETER
● MOISTURE CONTENT %



DEPTH FEET	MATERIAL DESCRIPTION	DEPTH	TESTING	SAMPLE TYPE SAMPLE ID	COMMENTS
0.0	TOPSOIL/TILL ZONE	0.0			
	Brown, silty SAND (SM); low plasticity; fine to medium sand; moist	1.0	ATT	S-1	LL = 19 PL = 18 PI = 1 02/20/21
2.0	becomes wet				Seepage at 1.5 feet bgs
4.0					
6.0				S-2	
8.0					
	Brown to gray, poorly graded SAND (SP-SM) with silt; non-plastic; fine to medium sand; wet	9.0			
10.0			P200	S-3	P200 = 8%
	Final depth 10.5 feet bgs; test pit backfilled with excavated material to existing ground surface.	10.5			
12.0					



LOGGED BY: D. Eibert
COMPLETED: 2/20/2021

EXCAVATED BY: Dan J. Fischer Excavating, Inc.
EXCAVATION METHOD: Mini Excavator with 24" Bucket

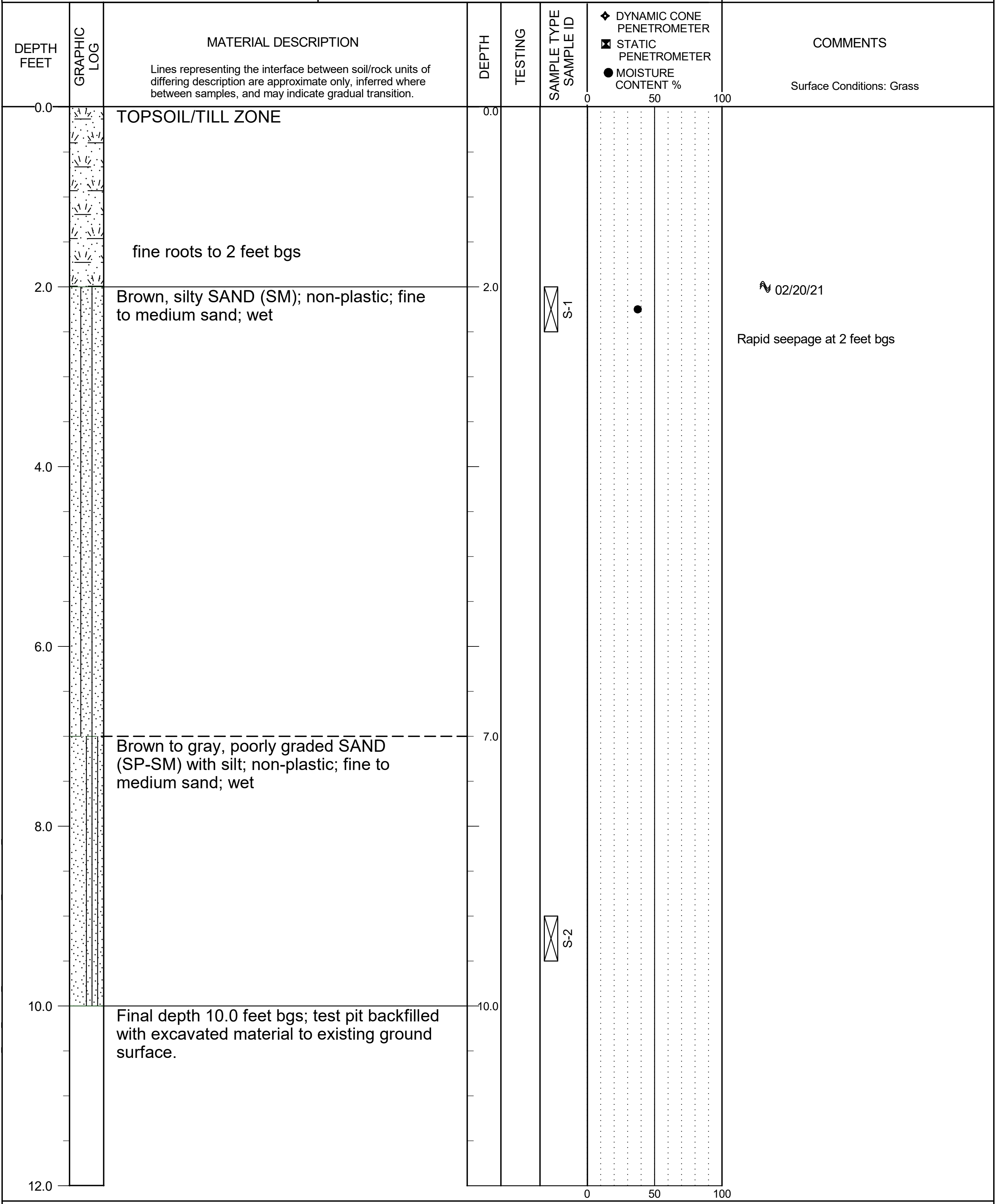
FIGURE A17

48 BED COMMUNITY RTF
VANCOUVER, WASHINGTON

PBS PROJECT NUMBER:
73200.011

TEST PIT TP-15

APPROX. TEST PIT TP-15 LOCATION:
(See Site Plan)
Lat: 45.73704 Long: -122.61733



Lines representing the interface between soil/rock units of differing description are approximate only, inferred where between samples, and may indicate gradual transition.

Surface Conditions: Grass

0.0 – TOPSOIL/TILL ZONE

fine roots to 2 feet bgs

2.0 – Brown, silty SAND (SM); non-plastic; fine to medium sand; wet

S-1

02/20/21

Rapid seepage at 2 feet bgs

7.0 – Brown to gray, poorly graded SAND (SP-SM) with silt; non-plastic; fine to medium sand; wet

S-2

10.0 – Final depth 10.0 feet bgs; test pit backfilled with excavated material to existing ground surface.

LOGGED BY: D. Eibert
COMPLETED: 2/20/2021

EXCAVATED BY: Dan J. Fischer Excavating, Inc.
EXCAVATION METHOD: Mini Excavator with 24" Bucket

FIGURE A18
Page 1 of 1

48 BED COMMUNITY RTF
VANCOUVER, WASHINGTON

PBS PROJECT NUMBER:
73200.011

TEST PIT TP-16

APPROX. TEST PIT TP-16 LOCATION:
(See Site Plan)

Lat: 45.73709 Long: -122.61645

MATERIAL DESCRIPTION

Lines representing the interface between soil/rock units of differing description are approximate only, inferred where between samples, and may indicate gradual transition.

◆ DYNAMIC CONE PENETROMETER
☒ STATIC PENETROMETER
● MOISTURE CONTENT % (0–50–100)

Surface Conditions: Grass

DEPTH FEET	MATERIAL DESCRIPTION	DEPTH	TESTING	SAMPLE TYPE SAMPLE ID	COMMENTS
0.0	TOPSOIL/TILL ZONE	0.0			
	roots to 1.5 feet bgs				
	Brown, silty SAND (SM); low plasticity; fine to medium sand; moist	1.5			
2.0	becomes wet				02/02/21
					Seepage at 2.5 feet bgs
				S-1	
4.0					
					Caving below 5 feet bgs
			P200	S-2	P200 = 37%
6.0					
8.0					
	Gray to brown, poorly graded SAND (SP-SM); non-plastic; fine sand; wet	8.5			
				S-3	
10.0					
	Final depth 10.5 feet bgs; test pit backfilled with excavated material to existing ground surface.	10.5			
12.0					

LOGGED BY: D. Eibert
COMPLETED: 2/20/2021

EXCAVATED BY: Dan J. Fischer Excavating, Inc.
EXCAVATION METHOD: Mini Excavator with 24" Bucket

FIGURE A19
Page 1 of 1

48 BED COMMUNITY RTF
VANCOUVER, WASHINGTON

TEST PIT TP-17

PBS PROJECT NUMBER:
73200.011

APPROX. TEST PIT TP-17 LOCATION:
(See Site Plan)

Lat: 45.73698 Long: -122.61672

MATERIAL DESCRIPTION

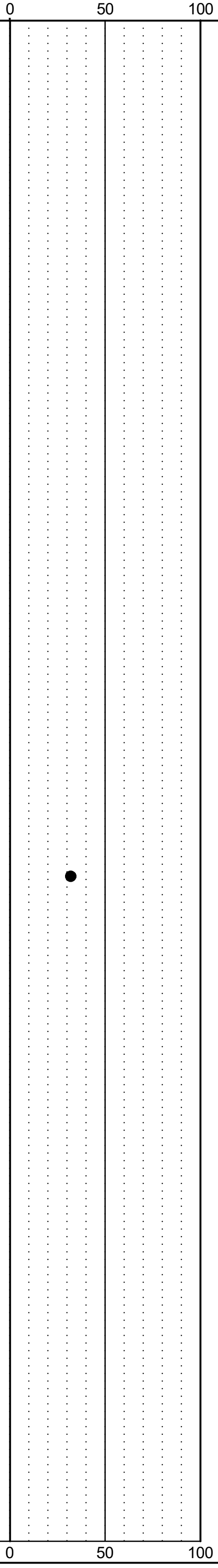
Lines representing the interface between soil/rock units of differing description are approximate only, inferred where between samples, and may indicate gradual transition.

◆ DYNAMIC CONE PENETROMETER
☒ STATIC PENETROMETER
● MOISTURE CONTENT %

COMMENTS

Surface Conditions: Grass

DEPTH FEET	MATERIAL DESCRIPTION	DEPTH	TESTING	SAMPLE TYPE SAMPLE ID	COMMENTS
0.0	TOPSOIL/TILL ZONE	0.0			
1.0	Brown, silty SAND (SM); low plasticity; fine to medium sand; moist	1.0		☒ S-1	≋ 02/20/21
2.0	becomes wet				Rapid seepage at 1.5 feet bgs
6.5				☒ S-2	
10.5	Final depth 10.5 feet bgs; test pit backfilled with excavated material to existing ground surface.	10.5			



LOGGED BY: D. Eibert
COMPLETED: 2/20/2021

EXCAVATED BY: Dan J. Fischer Excavating, Inc.
EXCAVATION METHOD: Mini Excavator with 24" Bucket

FIGURE A20
Page 1 of 1

TEST PIT LOG - 1 PER PAGE 73200.011 B1-3 TP1-17 20210217.GPJ PBS DATATMPL GEO.GDT PRINT DATE: 4/12/21:RPG

PBS Engineering and Environmental Inc.
4412 S Corbett Avenue
Portland, Oregon
http://www.pbsusa.com

CPT-1
Total depth: 56.10 ft, Date: 11/18/2020

Project: **73200.011 48 Bed Community RTF**
Location: **Vancouver, Washington**

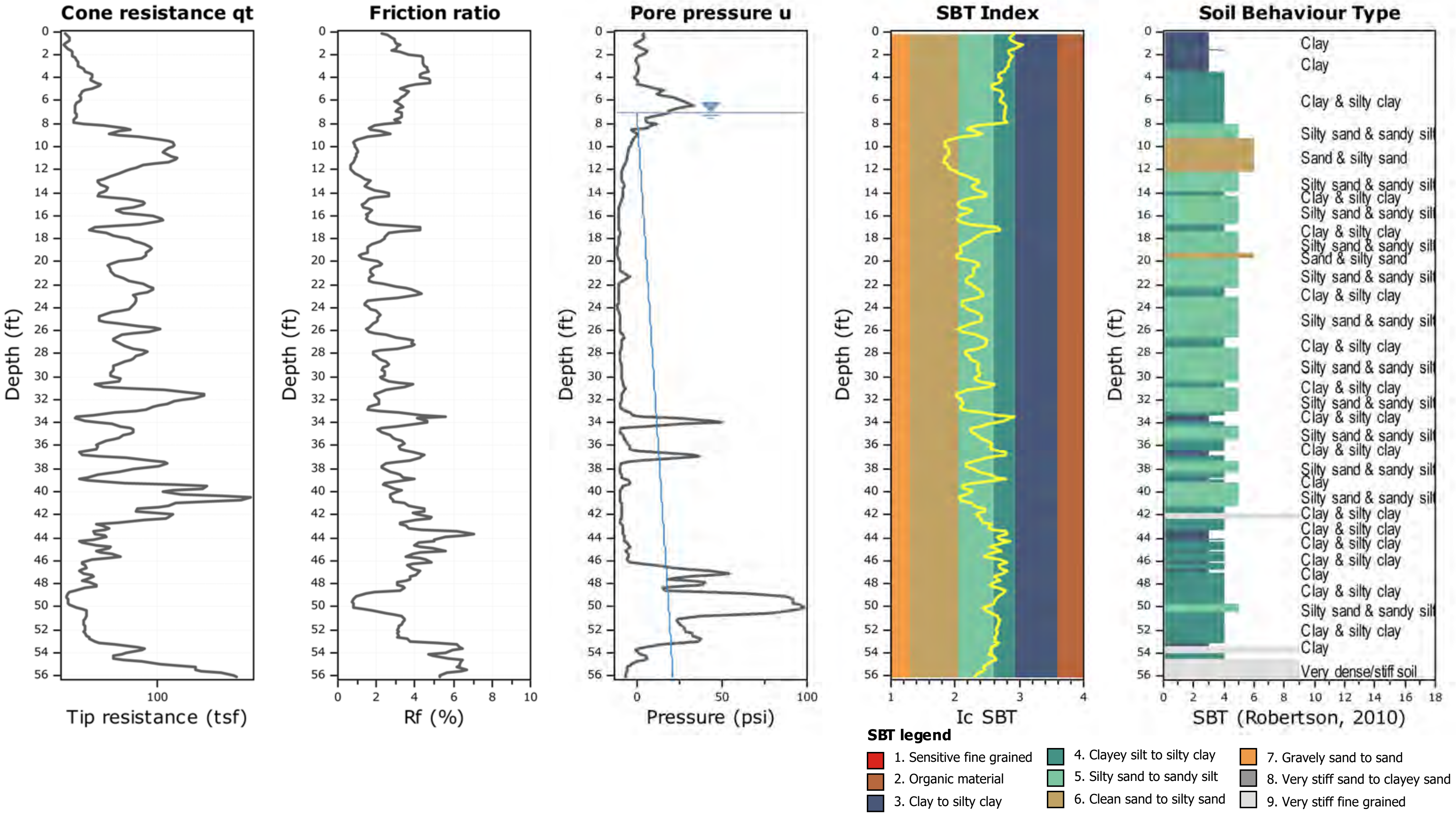


FIGURE A21

PBS Engineering and Environmental Inc.
4412 S Corbett Avenue
Portland, Oregon
http://www.pbsusa.com

CPT-2
Total depth: 50.53 ft, Date: 11/18/2020

Project: **73200.011 48 Bed Community RTF**
Location: **Vancouver, Washington**

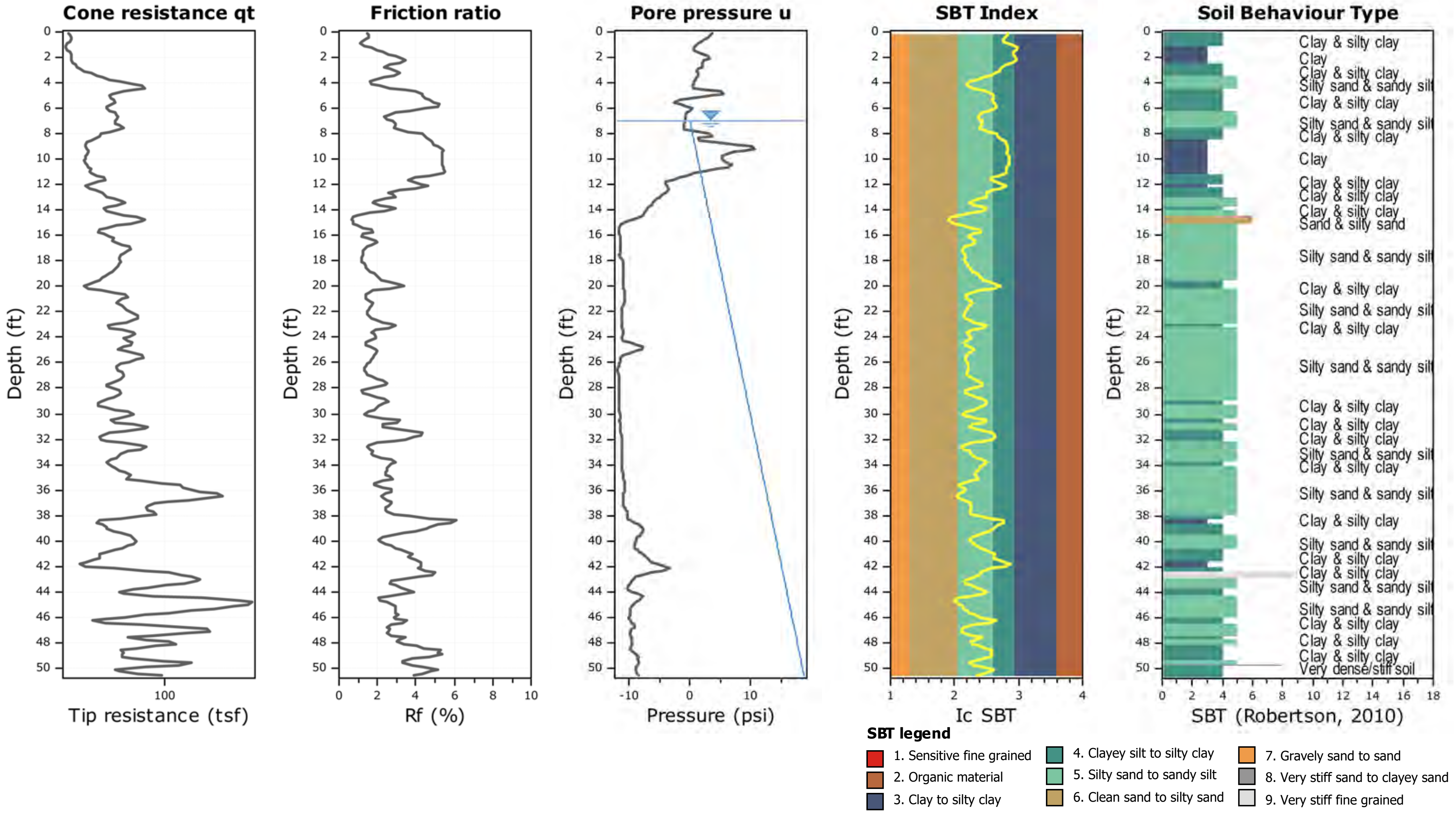


FIGURE A22

PBS Engineering and Environmental Inc.
4412 S Corbett Avenue
Portland, Oregon
http://www.pbsusa.com

CPT-3
Total depth: 50.53 ft, Date: 11/18/2020

Project: **73200.011 48 Bed Community RTF**
Location: **Vancouver, Washington**

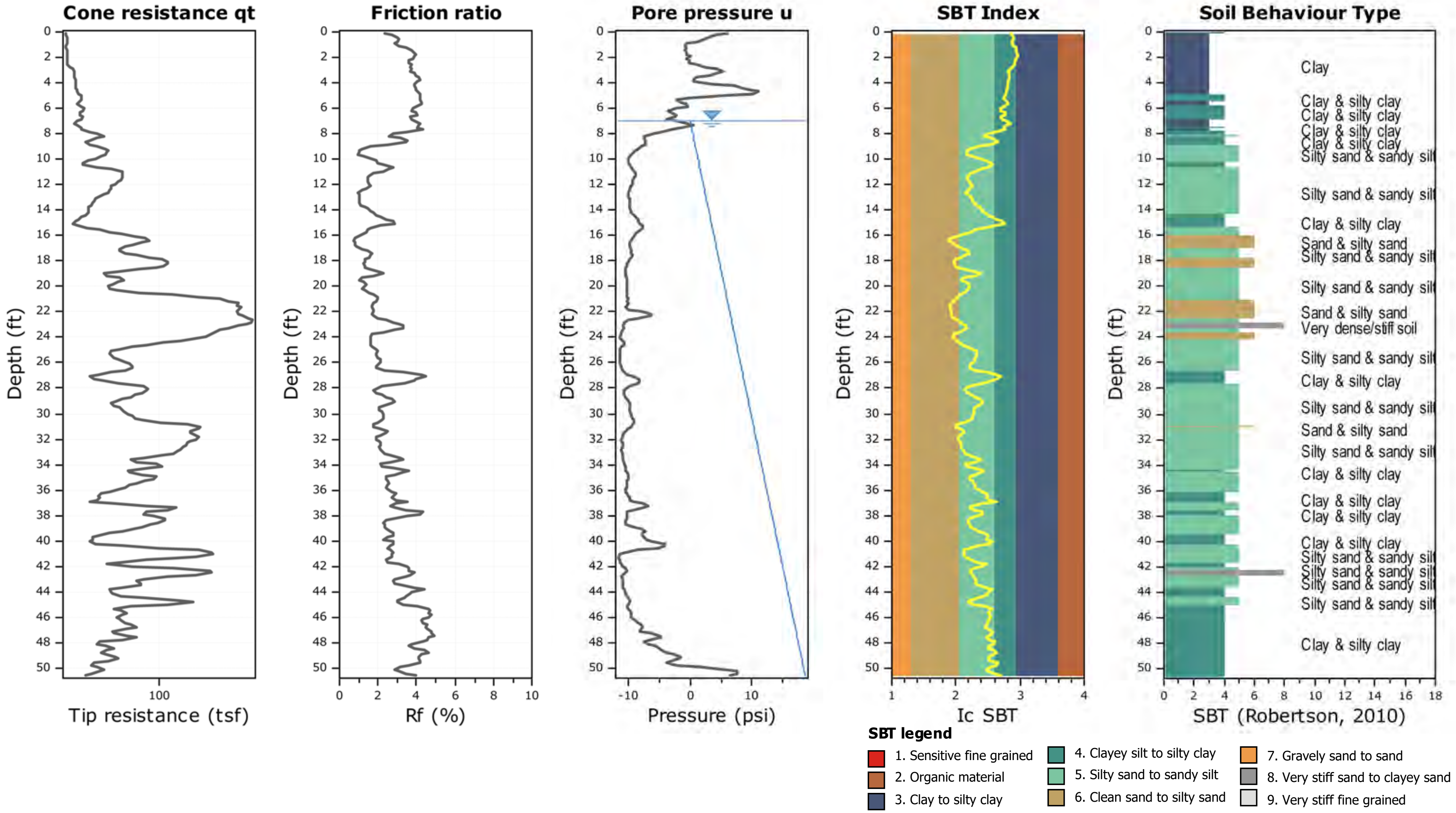
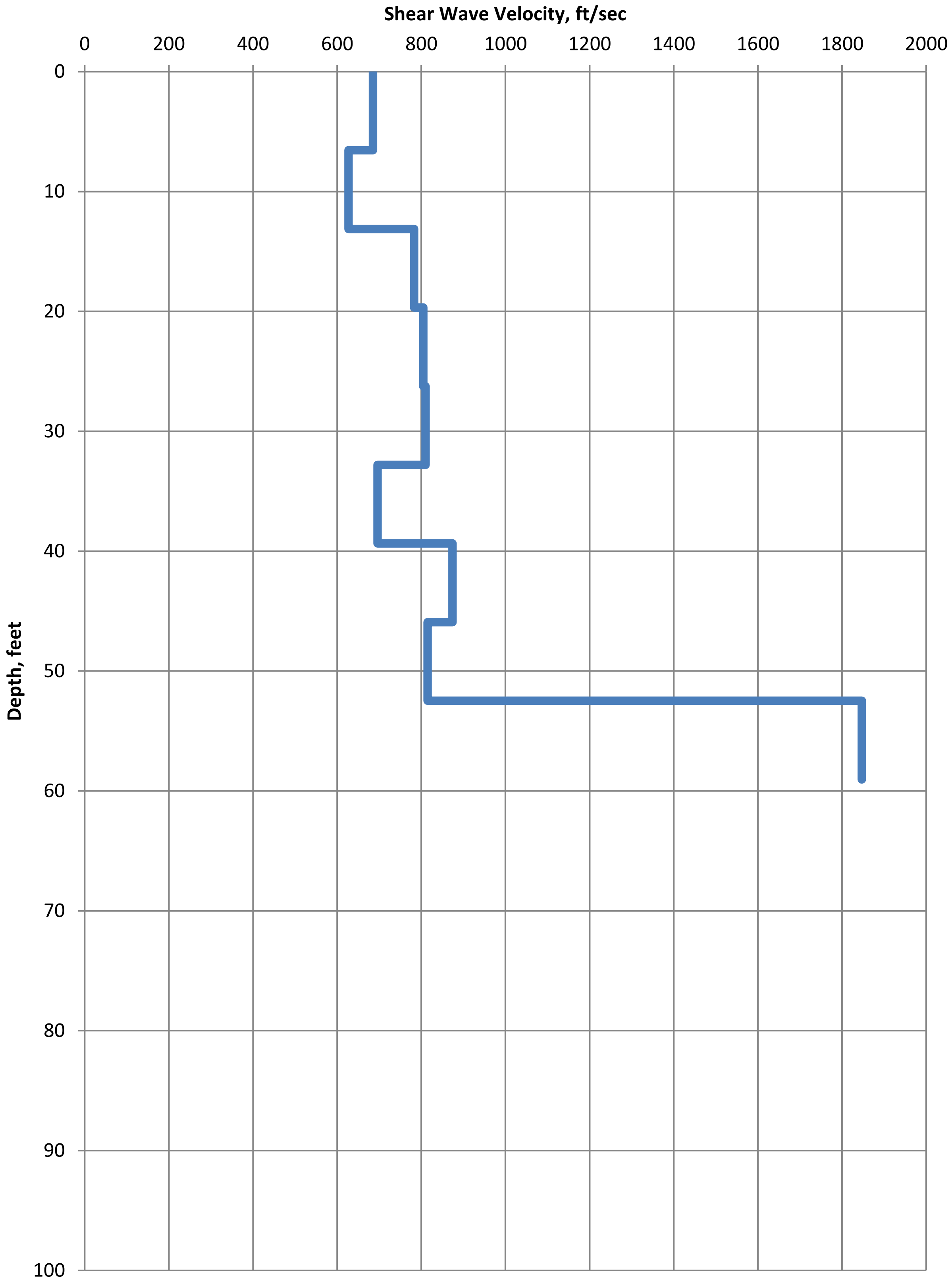


FIGURE A23



SHEAR WAVE VELOCITY PROFILE

48 BED COMMUNITY RTF
VANCOUVER, WASHINGTON

PBS

APR 2021
73200.011

FIGURE
A24

Appendix B

Laboratory Testing

Appendix B: Laboratory Testing

B1 GENERAL

Samples obtained during the field explorations were examined in the PBS laboratory. The physical characteristics of the samples were noted and field classifications were modified where necessary. During the course of examination, representative samples were selected for further testing. The testing program for the soil samples included standard classification tests, which yield certain index properties of the soils important to an evaluation of soil behavior. The testing procedures are described in the following paragraphs. Unless noted otherwise, all test procedures are in general accordance with applicable ASTM standards. "General accordance" means that certain local and common descriptive practices and methodologies have been followed.

B2 CLASSIFICATION TESTS

B2.1 Visual Classification

The soils were classified in accordance with the Unified Soil Classification System with certain other terminology, such as the relative density or consistency of the soil deposits, in general accordance with engineering practice. In determining the soil type (that is, gravel, sand, silt, or clay) the term that best described the major portion of the sample is used. Modifying terminology to further describe the samples is defined in Table A-1, Terminology Used to Describe Soil, in Appendix A.

B2.2 Moisture (Water) Contents

Natural moisture content determinations were made on samples of the fine-grained soils (that is, silts, clays, and silty sands). The natural moisture content is defined as the ratio of the weight of water to dry weight of soil, expressed as a percentage. The results of the moisture content determinations are presented on exploration logs in Appendix A and on Figure B2, Summary of Laboratory Data, in Appendix B.

B2.3 Atterberg Limits

Atterberg limits were determined on select samples for the purpose of classifying soils into various groups for correlation. The results of the Atterberg limits test, which included liquid and plastic limits, are plotted on Figure B1, Atterberg Limits Test Results, and on the explorations logs in Appendix A where applicable.

B2.4 Grain-Size Analyses (P200 Wash)

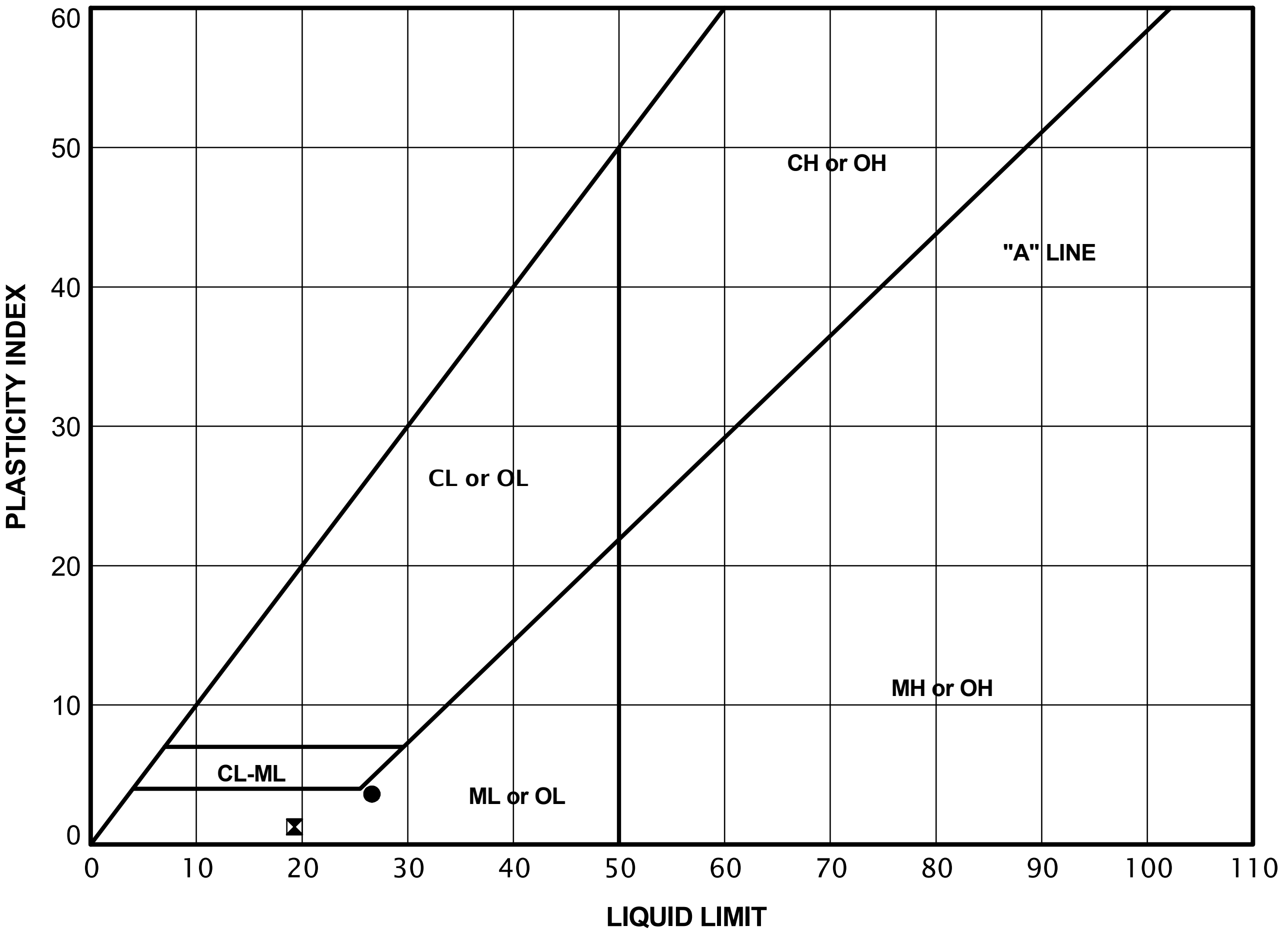
Washed sieve analyses (P200) were completed on samples to determine the portion of soil samples passing the No. 200 Sieve (i.e., silt and clay). The results of the P200 test results are presented on the exploration logs in Appendix A and on Figure B2, Summary of Laboratory Data, in Appendix B.

ATTERBERG LIMITS TEST RESULTS

48 BED COMMUNITY RTF
VANCOUVER, WASHINGTON

PBS PROJECT NUMBER:
73200.011

TEST METHOD: ASTM D4318



KEY	EXPLORATION NUMBER	SAMPLE NUMBER	SAMPLE DEPTH (FEET)	NATURAL MOISTURE CONTENT (PERCENT)	PERCENT PASSING NO. 40 SIEVE (PERCENT)	LIQUID LIMIT	PLASTIC LIMIT	PLASTICITY INDEX
●	B-3	S-8	30.0	28.7	NA	27	23	4
☒	TP-14	S-1	1.0	24.9	NA	19	18	1

ATTERBERG LIMITS 73200.011_B1-3_TP1-17_20210217.GPJ PBS_DATATMPL_GEO.GDT PRINT DATE: 3/2/21:RPG

FIGURE B1

PBS

SUMMARY OF LABORATORY DATA

48 BED COMMUNITY RTF
VANCOUVER, WASHINGTON

PBS PROJECT NUMBER:
73200.011

SAMPLE INFORMATION				MOISTURE CONTENT (PERCENT)	DRY DENSITY (PCF)	SIEVE			ATTERBERG LIMITS		
EXPLORATION NUMBER	SAMPLE NUMBER	SAMPLE DEPTH (FEET)	ELEVATION (FEET)			GRAVEL (PERCENT)	SAND (PERCENT)	P200 (PERCENT)	LIQUID LIMIT (PERCENT)	PLASTIC LIMIT (PERCENT)	PLASTICITY INDEX (PERCENT)
B-1	S-2	5	207.0	33.9				41			
B-1	S-7	25	187.0	35.7				65			
B-2	S-2	5	205.0	34.6				23			
B-2	S-8	30	180.0	33.7				65			
B-3	S-6	20	188.0	33.6				61			
B-3	S-8	30	178.0	28.7					27	23	4
TP-1	S-3	8		18.3				12			
TP-2	S-1	4		22.9				42			
TP-3	S-1	4		21.3				36			
TP-5	S-1	4		28.3							
TP-5	S-3	9.5		46.3				11			
TP-6	S-1	4		24.9							
TP-7	S-2	5		25.4				31			
TP-7	S-3	7		19.1				28			
TP-8	S-2	5		29.2				36			
TP-9	S-2	7		33.9				48			
TP-10	S-1	1		35.7							
TP-11	S-2	6		31.4							
TP-12	S-1	5		30.9							
TP-13	S-2	8		30.3				30			
TP-14	S-1	1		24.9					19	18	1
TP-14	S-3	10		31.5				8			
TP-15	S-1	2		37.4							
TP-16	S-2	5.5		35.5				37			
TP-17	S-2	6.5		31.9							

FIGURE B2